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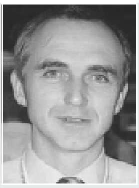



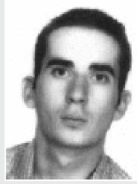

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Procedures for the assessment of highway structures

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Bridges, earth-retaining walls and buried structures make up a substantial proportion of the fixed assets of the land-based transportation infrastructure within Europe. Little work has been done on the development of documents covering the assessment of highway structures compared to the design of new structures. This paper describes an approach to assessment developed through working groups 4 and 5 of the European Cooperation in Science and Technology (Cost) Action 345, entitled procedures required for assessing highway structures. This action was supported by the European Commission and involved experts from 16 European countries. The ICE trust fund has supported a study of a road bridge in Vienna to demonstrate the applicability and potential benefits of the approach developed through Cost 345. The approach is similar to that used in the UK in that there are five levels of assessment of increasing complexity and reliability, but there are a number of differences. This paper describes the approach developed through Cost 345 with a view to opening the debate on the need for a code of practice for assessment that facilitates the use of site-specific loading.

1. INTRODUCTION

Highway structures need to cater for an increasing demand in transport capacity while in many cases deteriorating through, for example, corrosion and excessive deformation. As a result, in-service structures require assessment and a prioritisation of the measures necessary to ensure their structural integrity and safety. It is generally accepted that the use of design standards for assessment is overconservative and can lead to unnecessary replacement or strengthening of existing structures with all the attendant costs of traffic delays. In the design phase, loading conditions may be overestimated and structural strength underestimated to cater for the inherent uncertainties associated with in-service conditions. While the cost of providing this enhanced level of safety is marginal at the design stage, the same cannot be said for assessment, where overconservatism can lead to considerable unnecessary expenditure.

The recent development of probabilistic and reliability-based assessment approaches has contributed towards the establishment of more rational assessment procedures. From 1999 to 2002, the European Commission's Cooperation in Science and Technology (Cost) programme supported the action entitled **procedures required for assessing highway structures**, which involved experts from 16 countries. The main objectives of the action, known as Cost 345,[1] were to obtain information and extend knowledge on

(*a*) the current European practice on inspection, assessment, maintenance and repair of in-service highway structures (i.e. bridges, culverts, retaining walls, and tunnels)
(*b*) the age and condition of the stock of highway structures in Europe.

The output of Cost 345 was targeted at engineers, network managers and authorities to assist them in selecting the appropriate methodology for assessing the safety and serviceability of highway structures. This paper reports on the issues addressed by Working Groups 4 and 5 of Cost 345, namely numerical methods for assessing the safety and serviceability of highway structures.

In line with the UK standard,[2,3] the Cost approach recommends that the level of assessment should vary according to the time available to the engineer, the degree of sophistication of the analysis and the data available from the structure being assessed. Higher levels of assessment require measurements of material strength and in-service loading conditions as well as complex modelling and analysis. However, the Cost 345 approach places a particular emphasis on the use of site-specific load data. A practical demonstration of the approach adopted by Cost 345 has been supported by the ICE trust fund in a test on a road bridge in Vienna, where site-specific traffic data were collected and used to assess the reliability of the structure.[1,4]

It is the opinion of the authors that the Cost 345 recommendations provide considerable potential for savings in structure repair/replacement costs, particularly for less heavily trafficked roads where measurements can demonstrate that traffic loading is considerably less than that specified in design or assessment codes. The Cost 345 report may assist in the development of a European code of practice covering the assessment of highway structures, and this paper is intended to open the debate on the need for and content of such a code.

2. LEVELS OF ASSESSMENT

Visual inspections or structural monitoring (e.g. measurement of physical/chemical properties) can provide an indication of the degree of deterioration of a structure. In some cases, this preliminary assessment may find that an existing structure needs to be repaired (e.g. where the concrete is in poor condition). In other cases, even if the structure does not exhibit deterioration, it may need to be assessed for more onerous in-service loads in which case a numerical assessment will be undertaken to judge whether the level of risk is acceptable or not. A simple analysis would be cost-effective where it demonstrated that the level of risk was acceptable, but where it was unacceptable the engineer should introduce more advanced methods of assessment. Indeed the engineer may proceed to a more elaborate assessment in anticipation of future deterioration or increases in loading, to facilitate the bridge management process. Hence, similar to guidelines in the UK[5] and the European Bridge Management in Europe (BRIME) project,[6] Cost 345 classifies assessment in five distinct levels, numbered I to V, with level I being the simplest and level V the most sophisticated, as follows.

(*a*) *Level I assessment*. Only simple analysis methods are necessary and full partial safety factors from the assessment standards are used to give a conservative estimate of load capacity.

(*b*) *Level II assessment*. More refined analysis and better structural idealisation than that in level I is employed. Characteristic strengths for the structural materials are determined based on existing available data. Full partial safety factors are again used.

(*c*) *Level III assessment*. Unlike level II, site-specific loading and/or material properties are determined from new tests on the structure. Full partial safety factors are again used.

(*d*) *Level IV assessment*. Lower levels of assessment depend on implicit levels of safety where reliability is based on the majority of structures of the type concerned. On the other hand, level IV takes into account any additional safety characteristic of the structure being assessed by allowing modified safety criteria determined through rigorous reliability analysis or by judgmental changes to the partial safety factors.

(*e*) *Level V assessment*. Reliability analysis is applied to a particular structure. This analysis requires probabilistic data for all the variables defined in the loading and strength equations in addition to specialist knowledge and expertise.

3. LOAD MODELLING

An existing bridge has to carry the same type of loads as a new one. From an assessment point of view, these loads can be classified into time invariant (e.g. dead and superimposed dead load) and time variant (e.g. traffic, wind, earthquake and temperature loading). Some other loads can be initially time variant but asymptotically time invariant at some point (i.e. differential settlement, earth pressures, creep and shrinkage effects, etc.).

3.1. Time-invariant loads

Time-invariant loads are those that may reasonably be expected to remain the same for the lifetime of the structure. Compared to the design stage, at assessment stage these loads may have altered due to the effects of the construction process and subsequent life of the structure. As a result, calculations can often be assisted by on-site measurements which allows them to be determined more accurately (e.g. measurement of the actual thickness of the asphalt layer). As a result, the load and resistance models can be updated while maintaining the required safety for the structure. Measurement of density and element size may also justify the use of a lower value of a particular partial factor for assessment than that adopted for design.

3.2. Time-variant loads

Time-variant loads are those that can be modelled as stochastic variables. When assessing a structure, representative records of traffic, wind, earthquake (if relevant) and temperature records may be available. Hence, the characteristic load effects (i.e. values of bending moment, shear force, etc. with specified probability of exceedance), may be predicted more accurately.

3.2.1. Statistical modelling. Time-variant live loads, such as traffic, wind and temperature effects represent random phenomena and as such require statistical modelling to determine the magnitude of their characteristic effects. Extreme value distributions, such as those contained within the Gumbel family (i.e. Gumbel, Frechet and Weibull), are fitted to measured data recorded over a period of time. Subsequent extrapolation of these fitted distributions for a specified return period, yields the characteristic value. The difference in distribution contained within the Gumbel family lies in their tail behaviour—that is, whether they are bounded or unbounded in the extreme. In modelling measured data for prediction of characteristic extremes, care should be taken to ensure that the most appropriate distribution is selected. One way of doing this is by plotting the data on the probability paper relating to the chosen distribution. Standard transformation functions are employed to determine relative plotting positions for the recorded data on the probability paper. The degree of linearity of the plotted data and the closeness of fit of the chosen distribution to the data, reflects the accuracy of the distribution. For example, Fig. 1 shows the probability paper plots for the Gumbel and Weibull extreme value distributions respectively, for bending moment at the centre of a single-span, two-lane, 20 m long bridge. In the figure the parameters λ and η are the respective limit and dimensionless scaling parameters relating to the distributions. Differences in the mathematical formulation of the Gumbel and Weibull distributions are apparent in the relationship between the transformed data, plotted on probability paper, and the best-fit extreme value distribution. Care should be taken in selecting the appropriate distribution for extrapolation.

3.2.2. Load monitoring data required for assessment. The duration of the measurements used for extreme value modelling depends upon the effect being determined. For wind and temperature data, maximum and minimum values of the particular effect over a representative period of time (e.g. 50 years) and for a specified sampling frequency (e.g. monthly) should be collected. Traffic weigh-in-motion (WIM) data can be obtained by mounting sensors in the road pavement or on an existing bridge structure and estimating the corresponding static loads using appropriate algorithms. There is considerable variation in the distributions of gross vehicle weight (GVW)

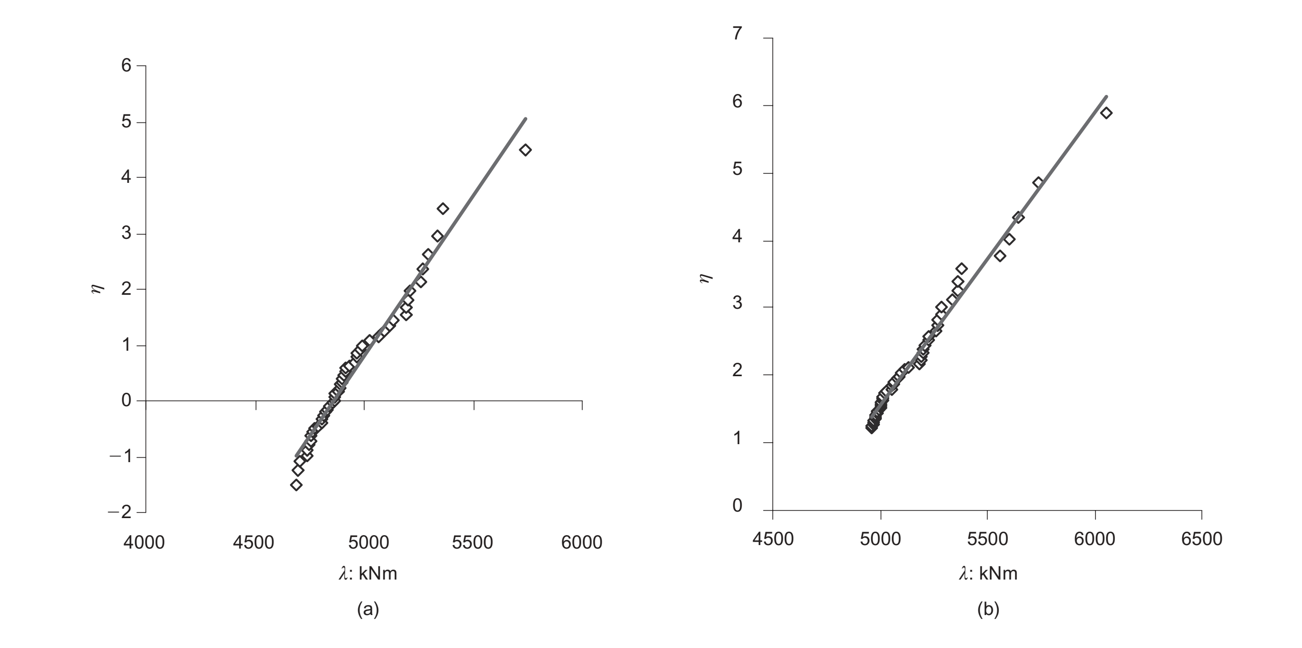

Fig. 1. Extreme value approximations for simply supported bridge and bending moment at centre of span (total span 20 m): (a) Gumbel I and (b) Weibull

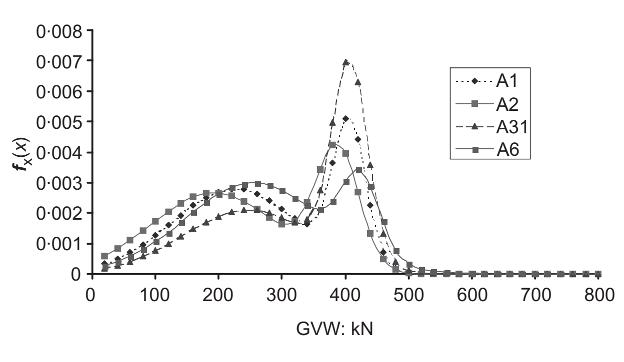

Fig. 2. Comparison of gross vehicle weight distributions at various sites on the French motorway network

between sites, as can be seen in Fig. 2. It is therefore important to collect data appropriate to the bridge being assessed.

It is clearly desirable to collect as much data as possible, but one or two weeks of continuously recorded data may be sufficient for the purposes of assessment.[7,8] It is important to attempt to ensure that these data are representative and so, in scheduling a measurement period, consideration should be given to seasonal variation patterns.

The Cost 345 report does not specify the required accuracy of WIM data. However, some guidance is given by Jacob *et al.*[9] These authors specify the required accuracy with reference to the Cost 323 WIM specification.[10,11] Bridge loading is not sensitive to WIM system accuracy and a system with accuracy as low as C(15) is deemed to be sufficient. This corresponds to about 95% of GVWs (the exact percentage depends on test conditions) being within 15% of the exact static value.[11]

3.2.3. Traffic load simulation for assessment. The characteristics of the vehicles that traverse a bridge vary widely with respect to their gross weight, axle spacing, distribution of load to axles, location in lane, velocity and in the likelihood of multiple presence of vehicles on the structure both longitudinally and transversely. In level I and II assessments, extremes of normal traffic are represented by notional load models. Some countries have assessment standards with notional load models that are generally less onerous than those specified for new structures. In countries that do not have specific standards for assessment, the notional load model for new structures may be used. Such codes are generally conservative in the interests of simplicity and given the cost insensitivity of new structures to loading. Assessment load models can be developed in European countries where the Eurocode has been calibrated for national conditions and where the data used for that process are available. It has been established in such studies that the notional load model is considerably more conservative for some load effects and some road classes than others,[12] a phenomenon that could be exploited to provide reduction factors for the Eurocode model.

For level III to level V assessments, site-specific load modelling is allowed in the Cost 345 report. A number of approaches can be used to convert basic WIM statistics into estimates of characteristic load effects. The CASTOR-LCPC program[13] uses WIM data directly to generate traffic streams with axle spacings, weights, etc. Influence surfaces are used to convert the traffic streams into load-effect time histories from which load variables (such as maximum-per-day histograms) can be determined. This approach has the advantage that few assumptions regarding the nature of the data are necessary. It is applicable to short-span bridges but, given the relative paucity of data in the tail of the truck GVW distribution, it requires a considerable amount of data to give repeatable results. The direct simulation approach is unlikely to include enough examples of traffic congestion to accurately simulate the loading events that govern long-span bridges.

Variable	Notation	Bias	COV
Elastic limit of structural steel (welded)[16]	f_y	1·25	0·08
Elastic limit of structural steel (rolled)[16]	f_y	0·99	0·05
Compressive strength of concrete (20–40 MPa)[17]	f'_c	1·31–1·19	0·14–0·09
Tensile strength of concrete (20–40 MPa)[17]	f_t	1·47–1·28	0·18–0·16
Modulus of elasticity of concrete[18]	E_c	1·18	0·10
Tensile strength of reinforcing steel (400 MPa)[17]	f_y	1·22	0·08
Modulus of elasticity of reinforcing steel[17]	E_s	1	Modelled as deterministic[17]

Table 1. Examples of systematic (bias) and random uncertainties in material properties[16,17]

An alternative to direct simulation is to form a histogram of GVW and to fit a distribution to this. The distribution is subsequently used with Monte Carlo simulation, to generate 'typical' weights for vehicles in a simulated traffic stream. The histogram can be used directly to generate weights but this, like the direct approach described above, suffers from a paucity of data in the tail region. Many authors have fitted bi- or tri-normal statistical distributions to GVW histograms;[14] the histograms of Fig. 2, for example, fit well to a bi-normal distribution. This is easy to implement but Getachew and O'Brien[15] have suggested that a good fit to the distribution overall can result in a poor fit to the tail region, which has significant consequences. They recommend a combination of direct simulation from the histogram where there is sufficient data and the fitting of a normal distribution to the data in the tail region. For short-span bridges, arch bridges and culverts, the weight of the individual axle or axle group can be more important than the GVW and these need to be represented accurately in simulations.

A major shortcoming of existing codes lies in the manner in which they have taken account of the dynamic amplification of the static effect resulting from the interaction between the moving vehicle and the structure. In the Eurocode, dynamic multipliers are applied to characteristic extremes (i.e. load-effect values with a specified probability of exceedance) determined from simulation. Such an approach ignores the combination of probabilities from two independent phenomena: the meeting of heavy vehicles on a bridge and the dynamic interaction between the vehicles and the bridge. As the worst static and dynamic effects do not generally occur for the same loading case, site-specific maximum design load effects determined from measured traffic data and experimental bridge dynamic characteristics could lead to significant savings when assessing a structure.

Additionally, the structural resistance models can be updated through an accumulation of knowledge of the loads that the structure has successfully carried to date. If a bridge has survived a number of years of service, its resistance is higher than any of the prior imposed loads. The improvement of bridge reliability with proven service can be taken into account in a level IV assessment. However, this can be difficult to achieve in practice without a comprehensive knowledge of the load history of the structure.

4. MATERIAL MODELLING

Material properties vary both spatially and temporally. There are spatial variations due to the different combinations of material components within each location (e.g. different combinations of aggregate, cement and water in the case of concrete) and temporal variations due to the loading and physical processes in the materials (e.g. hydration process in concrete that increases stiffness and strength). The difference between the material properties of the test specimens and of that forming the structure must be considered. Table 1 gives examples (from buildings) of systematic (bias) and random (coefficient of variation) uncertainties found in some engineering material properties.[16,17]

The Cost 345 report discusses a number of issues associated with concrete and steel modelling. A number of mathematical models have been developed to determine the *in situ* compressive strength of concrete.[18–21] When determining the values of material properties to be used in the assessment of an existing structure, the difference between test values and *in situ* material properties as well as the effects of compliance controls must be considered using probabilistic methods. Normal or lognormal distributions are typically used to represent the basic compressive strength of concrete[22] and normal or beta distributions can be used to represent yield strength of steel reinforcement properties.[18,22] Mathematical models of concrete properties can also be improved by considering the degree of quality control[18,23] and the drying shrinkage of concrete.[24]

Partial safety factors from assessment standards (lower than at the design stage) can be used for level I, but characteristic strengths for materials can be based on existing data (i.e. mill certificates and results of *in situ* testing from the same or a similar structure) for level II with allowance for the differences between, for example, cores and characteristic cube strengths. Information from load tests can be used for level III or higher. Level IV uses modified partial safety factors to account for any additional safety characteristics specific to the structure being assessed and level V uses structural reliability analysis instead of partial safety factors.

5. STRUCTURAL RESPONSE MODELLING

The assessment of a highway structure requires the calculation of the response of a mathematical model of the structure to a complete range of loading conditions. The method of analysis to be used will depend on the behaviour of the structural material, structural geometry and boundary conditions, as well as the nature of the applied load.

5.1. Methods of structural analysis

Separate or interdependent mathematical models of the structure and the soil can be established to determine the structural response. Hence a particular model for a structure will be influenced by the assumptions adopted for the foundation and the soil. If it is ensured that the ground can sustain the loading with acceptable displacements or provide adequate stiffness, soil–structure interaction can be ignored in low-level studies (in bridges, piled foundations have often been employed to provide relatively rigid foundations and allow an analysis of the structure in isolation). Cost 345 reviews a number of available techniques to model the structural response according to ultimate and serviceability limit states. Perspectives are given on a reliability-based design/assessment approach and on empirical, algebraic and numerical (e.g. finite-element) methods of analysis, the number of dimensions of the structural model (frame and spatial analysis), the behaviour of the structural material (elastic or plastic), the magnitude of the displacements with respect to the original geometry (linear or non-linear), the characteristics of the section (cracked or uncracked reinforced concrete), the nature of the applied load (static, dynamic, impact, fatigue) and the definition of the structure (in deterministic or probabilistic terms).

An assessment at level I is carried out with traditional methods of structural analysis while assessment at higher levels involves more refined methods of analysis. Traditional methods of structural analysis are based on one- or two-dimensional models with elastic materials, geometric linearity and static loads. Other more rigorous techniques allow for three-dimensional modelling, a variety of non-linear response actions and dynamics. The number of variables involved in the modelling process increases with the level of assessment. Ideally in higher levels of assessment, the method of analysis should take account of all the significant aspects of the structural response to loads and imposed displacements. The same methods of analysis can be used for level II and higher levels, but specific material properties and loading can be included in higher levels. All categories are summarised for bridges and culverts in Table 2. An assessment associated with complex mathematical modelling without field validation should be used with considerable caution.

5.2. Integration of field data and structural models

The original structure might have been altered not only due to ageing and the application of loads, but also to grouting, saddling, guniting or post-tensioning in maintenance and for upgrading programmes. It is always necessary to carry out a visual inspection of the structure. This inspection might reveal scouring of piers and/or abutment supports, cracks in a section of the structure, the quality and condition of the structural material, deformations of the profile, condition of the joints, damping devices, etc. The approach used (and the input values) to calculations can vary as a result of observation. Additionally, a number of reduction factors relating to the condition of the bridge can be defined according to observation. In the case of significant deformations, the cause should be determined and the new profile may need to be considered in the analysis.

The assessment of a structure might require a better idealisation of the structural response than one based on the observation of the visible portion of the structure. However, complex analytical tools can only be justified if a realistic assessment of the material properties and overall condition of the existing structure can be made through accurate field measurements. Tests on concrete include cover depth, rebound hammer, ultrasonics, impact echo, permeability, carbonation, thermography, radar, slot cutting, instrumented coring. Clearly, only some of these tests will serve to improve the assessment of load-carrying capacity. Testing for reinforcement corrosion includes the measurement of half-cell potentials, resistivity, chloride concentration and monitoring. Post-tensioning tendons can be tested with exploratory hole drilling, radiography, ultrasonics or through on-site monitoring. Other tests are related to the determination of *in situ* stress.[25]

Load testing can be used to improve the reliability of structural models for the serviceability limit state through measurement of static/dynamic effects and other performance measures, including the generation of cracks and the distribution of load. Such testing must be carried out with caution so as not to inflict damage to the structure. Thus the passage of heavily loaded trucks can be used to determine the on-site live-load behaviour of the structure and, by extrapolation, to predict the maximum stresses due to the traffic load. Typically, forced vibration or ambient vibration methods are used to determine the frequencies and mode shapes of vibration of a bridge. As tests at full scale are expensive and limited, scaled physical models using measurements from tests on the real structure, can also be used for assessment purposes. Garas *et al.*[26] verify through testing some of the methods of analysis at realistic scales that cannot be achieved in the laboratory.

Measurements can provide more realistic values for support stiffness, joint condition, restraints, behaviour of the cross-section, elastic properties of the structural material, behaviour of the foundation, fill and structural material density, road profile, etc. These characteristics can then be incorporated into the structural model. Optimisation techniques are commonly used for adjusting parameters of the structural models to fit with field measurements. The updated models can be used to more accurately predict and assess the behaviour of the structure under different static or dynamic loading conditions. In a structural reliability model, the uncertainties in the design parameters will be modelled probabilistically. The process of identifying the behaviour of a given structure is described by the ASCE Committee on Structural Identification of Constructed Facilities.[27]

6. RELIABILITY ANALYSIS

There are four main formats of reliability analysis, namely

(*a*) global safety factor format
(*b*) partial safety factor format
(*c*) reliability format
(*d*) socio-economical formats.

The global safety factor format, also referred to as permissible stress design, is not recommended for use in assessment. The other three methods are reviewed in the following subsections.

6.1. Partial safety factor format

Partial safety factors should reflect the knowledge of the uncertain parameters at the specified level of assessment, allowing for factors such as the quality of inspection, the extent and quality of on-site measurements, potential failure modes and possible consequences of failure. They can be calibrated using probabilistic methods and idealised reliability formats although, in most of the countries where a semi-probabilistic approach is applied, experience as well as

Structure type		Level of assessment 1	2	3	4	5
Bridges	Not skew beam	1-D or 2-D linear elastic (beam theory or plane frame analysis)	1-, 2- or 3-D linear or non-linear; elastic or plastic; allowing for cracking	2- or 3-D; linear or non-linear; elastic or plastic; grillage or FEM (upstand model if necessary); allowing for soil–structure interaction, cracking, and site-specific live loading and material properties	FEM analysis of specific details of the structure being assessed not considered in previous levels	Reliability analysis based on probabilistic models
	Not skew slab		2- or 3-D linear or non-linear; elastic or plastic; allowing for cracking; grillage or FEM (upstand model if necessary)			
	Not skew beam and slab					
	Not skew cellular					
	Skew, tapered and curved	1-D or 2-D simple grillage, linear elastic allowing for torsion				
	Arch	Empirical or 2-D linear elastic arch frame	2- or 3-D linear or non-linear; elastic or plastic; allowing for cracking			
	Cable stayed	2-D linear elastic with modified modulus of elasticity for the cables	2- or 3-D linear or non-linear; elastic or plastic; modelling cable sag more accurately			
Culverts	Rigid	Frame linear elastic	2- or 3-D FEM linear or non-linear; elastic or plastic; allowing for soil–structure interaction, cracking	2- or 3-D FEM, linear or non-linear; elastic or plastic; allowing for soil–structure interaction, cracking and site-specific loading and material properties		
	Flexible	Frame linear elastic allowing for soil–structure interaction (beam and spring)				
Earth-retaining walls		Simple equilibrium method of analysis	Beam, 2- or 3-D non-linear FEM on elastic foundation or elasto-plastic continuum	3-D non-linear FEM, allowing for soil constitutive models and site-specific loading and material properties		
Reinforced soil		Empirical models or 1-D linear elastic	2- or 3-D FEM of soil	2- or 3-D FEM of soil in combination with existing structure and site-specific loading and material properties		
Tunnels		Empirical models or beam-and-spring models (non-cohesive soil)	2- or 3-D FEM; linear or non-linear; elasto-plastic	3-D non-linear FEM with bedding, fracture planes, . . . and site-specific loading and material properties		

Table 2. Analysis methods recommended for each level of assessment

economic and political considerations have an influence on the values used. The partial safety factor format is the core of any modern design code and it is strongly recommended in Cost 345 for a general code of assessment of existing structures and for assessment up to level IV.

6.2. Reliability format

Partial safety factors are designed to cover a large number of uncertainties and may therefore not be representative for evaluating the reliability of a particular structure. The reliability format is also based on limit states, but unlike the semi-probabilistic partial safety factor format, it requires the calculation of the probability of failure with a specified reference period. This calculation involves the identification of all variables influencing the limit-state criteria, the statistical description of these variables, the derivation of the probability density and its moments for each basic variable, the calculation of the probability that the limit-state criterion is not satisfied, and the comparison of the calculated probability with a target level. Load demand, S, and capacity to resist load, R, are modelled as random variables in the formulation of the limit state. The difference between demand and capacity is known as the safety margin, M, where $M = (\boldsymbol{R} - \boldsymbol{S})$. The safety margin is normally distributed when $\boldsymbol{R}$ and $\boldsymbol{S}$ are normally distributed. The reliability index, β, is given by the ratio of the mean to the standard deviation of the safety margin. In many cases alternative distributions may be found to be appropriate to model the load and resistance variables.

The determination of the probability of failure for the limit state is a difficult task, except for linear limit states and Gaussian variables. The reliability index methods (e.g. first-order reliability method and second-order reliability method) and the simulation methods (e.g. Monte Carlo sampling and importance or directional sampling) are two techniques which allow the calculation of probability of failure for complicated functions. In most of the cases, appropriate software tools are necessary for structural reliability analysis.

A structure that can be proven to have a reliability index higher than the corresponding minimum value can be considered to be sufficiently safe. It is strongly recommended to use code calibration (partial safety factor format) to base levels I to IV assessment methods on a clear and documented reliability format. Thus, preliminary investigation at lower levels will open the way for a full reliability analysis at level V, in cases where the results of a partial safety factor format are believed to be too conservative.

Code	Target reliability index β		
ISO/CD 2394: 1998[20]	Relatively high cost of safety measure	Small consequences of failure	$\beta = 0{\cdot}0$
		Some consequences of failure	$\beta = 1{\cdot}5$
		Moderate consequences of failure	$\beta = 2{\cdot}3$
		Great consequences of failure	$\beta = 3{\cdot}1$
	Relatively moderate cost of safety measure	Small consequences of failure	$\beta = 1{\cdot}3$
		Some consequences of failure	$\beta = 2{\cdot}3$
		Moderate consequences of failure	$\beta = 3{\cdot}1$
		Great consequences of failure	$\beta = 3{\cdot}8$*
	Relatively low cost of safety measure	Small consequences of failure	$\beta = 2{\cdot}3$
		Some consequences of failure	$\beta = 3{\cdot}1$
		Moderate consequences of failure	$\beta = 3{\cdot}8$
		Great consequences of failure	$\beta = 4{\cdot}3$
JCSS[21] Ultimate limit state	Relatively large cost of safety measure	Minor consequences of failure	$\beta = 3{\cdot}1$
		Moderate consequences of failure	$\beta = 3{\cdot}3$
		Large consequences of failure	$\beta = 3{\cdot}7$
	Relatively normal cost of safety measure	Minor consequences of failure	$\beta = 3{\cdot}7$
		Moderate consequences of failure	$\beta = 4{\cdot}2$
		Large consequences of failure	$\beta = 4{\cdot}4$
	Relatively small cost of safety measure	Minor consequences of failure	$\beta = 4{\cdot}2$
		Moderate consequences of failure	$\beta = 4{\cdot}4$
		Large consequences of failure	$\beta = 4{\cdot}7$
Eurocode 1: 1993[29]	Serviceability	Design working life: bridges 100 years	$\beta = 1{\cdot}5$
		1 year	$\beta = 3{\cdot}0$
	Fatigue	Design working life: bridges 100 years	$\beta = 1{\cdot}5$ to $3{\cdot}8$
	Ultimate	Design working life: bridges 100 years	$\beta = 3{\cdot}8$
		1 year	$\beta = 4{\cdot}7$
NKB Report No. 36: 1978[30] ultimate limit state	Ductile with extra carrying capacity failure	Less serious failure consequences	$\beta = 3{\cdot}1$
		Serious failure consequences	$\beta = 3{\cdot}7$
		Very serious failure consequences	$\beta = 4{\cdot}2$
	Ductile without extra carrying capacity failure	Less serious failure consequences	$\beta = 3{\cdot}7$
		Serious failure consequences	$\beta = 4{\cdot}2$
		Very serious failure consequences	$\beta = 4{\cdot}7$
	Brittle failure	Less serious failure consequences	$\beta = 4{\cdot}2$
		Serious failure consequences	$\beta = 4{\cdot}7$
		Very serious failure consequences	$\beta = 5{\cdot}2$
CSA Ultimate limit state[31] $\beta = 3{\cdot}5 - (\Delta_E + \Delta_S + \Delta_I + \Delta_{PC}) \geqslant 2{\cdot}0$	Adjustment for element behaviour, Δ_E	Sudden loss of capacity, no warning	$\Delta_E = 0{\cdot}0$
		Sudden failure, no warning retention of post-failure capacity	$\Delta_E = 0{\cdot}25$
		Gradual failure, probable warning	$\Delta_E = 0{\cdot}5$
	Adjustment of system behaviour, Δ_S	Element failure leads to total collapse	$\Delta_S = 0{\cdot}0$
		Element failure probably does not lead to total collapse	$\Delta_S = 0{\cdot}25$
		Element failure leads to local failure only	$\Delta_S = 0{\cdot}5$
	Adjustment for inspection level, Δ_I	Component not inspectable	$\Delta_I = -0{\cdot}25$
		Component regularly inspectable	$\Delta_I = 0{\cdot}0$
		Critical component inspected by evaluator	$\Delta_I = 0{\cdot}25$
	Adjustment for traffic category, Δ_{PC}	All traffic categories except permit controlled	$\Delta_{PC} = 0{\cdot}0$
		Traffic category permit controlled	$\Delta_{PC} = 0{\cdot}6$

*For ultimate limit state analysis revise values. See code.

Table 3. Comparison of target reliability levels

6.3. Socio-economic format

Socio-economic formats are reliability formats where failure costs are introduced to determine the required probabilities of failure or reliability indices. This format can be based on decision-theory methods or on life-cycle cost methods. Thus, for example, the assessment procedure can take into consideration the fact that the failure of a retaining wall adjacent to a minor road has much lower consequences than the failure of a major bridge.

7. TARGET RELIABILITY LEVELS

The target reliability level is that level of reliability required to ensure the acceptable safety and serviceability of a structure. The authorities or bridge owners must specify the target reliability level. These levels can be explicitly or implicitly specified in a code. The target reliability level for assessment is different from that appropriate for the design stage due to

(*a*) economic considerations, which lead to the use of less conservative criteria for assessment as the increment in cost of upgrading an existing structure is much larger than that for increasing safety at the design stage
(*b*) social considerations, such as heritage values and disruption of occupants and activities caused by an intervention that do not affect the design of new structures
(*c*) sustainability considerations; for example, reduction of waste and recycling, more appropriate with the rehabilitation of existing structures.

Table 3 compares target lifetime reliability levels in various codes and standards currently in use. The engineer dealing with the assessment of an existing structure must decide among the available tables which of the values are most suited and best applied to the solution of the problem at hand as the estimated probability of failure associated with a project is very much a function of the costs as well as an understanding of the issues, modelling the data, etc. In the ISO/CD 13822: 1999,[28] the target reliability levels depend on the type of limit state examined as well as on the consequences of failure, and they range from a reliability index, β, of 2·3 for very low consequences of a structural failure to 4·3 for structures whose failure would have very severe consequences. In the ultimate limit state, a value of 4·3 would be suitable for most cases. The value β recommended by ISO 2394: 1998[20] and the JCSS[21] depends on the consequences of a structural failure as well as the costs of a safety enhancement measure. The JCSS target reliability index ranges from 3·1 to 4·7. In Eurocode 1,[29] β only depends on the type of limit state examined and it ranges from 1·5 to 4·7, while in the NKB report[30] the failure type and consequence is taken into account in the determination of β. The Canadian Standards Association (CSA) obtains β through an equation allowing for element and system behaviour, the inspectability and the traffic category, and it can take values between 2·00 and 3·75.[31]

8. CONCLUSIONS

This paper has described the procedures and numerical methods recommended by Working Groups 4 and 5 of Cost 345 for assessing the safety and serviceability of highway structures. As in the UK procedures,[2,3,5] there are five levels of assessment, varying from simple but conservative to complex but accurate. For each level of assessment, the processes by which load, material properties and structural response can be modelled, the reduction of partial safety factors through measurements of actual material strengths and loading conditions, and the target reliability levels for various codes and standards have been defined. Practical examples are also provided in the Cost 345 report. One of the aims of this action was to contribute to the continued safety and serviceability of the land transport fixed assets in Europe and elsewhere.

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