BULLETIN OF THE POLISH ACADEMY OF SCIENCES
TECHNICAL SCIENCES, Vol. 61, No. 1, 2013
DOI: 10.2478/bpasts-2013-0011

Uncertainties in the building process

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Abstract. Load and resistance have to be treated as random variables because of natural and man-made uncertainties. Statistical parameters, i.e. bias factors and coefficients of variation, are presented for natural effects of dead load, live load for buildings and bridges, and environmental loads. Man-made effects are more difficult to quantify as they include also human errors. For resistance, the uncertainties are considered as a combination of three factors: material, fabrication and professional. The statistical parameters are shown for structural steel, reinforced concrete, and prestressed concrete. These parameters allow for calculation of the reliability indices for structural components and assessment of the effect of uncertainties in load and resistance on the structural safety.

Key words: loads, resistance, bias factor, coefficient of variation, reliability index, limit states, human error.

1. Introduction

The structural design is based on prediction of load and resistance parameters, and selection of materials, dimensions and construction procedures so that the structure can performs adequately during the expected life time. However, a structural performance can be affected by uncertainties in the building process. Therefore, there is a need to review the causes and if possible quantify the degree of variation for load and resistance parameters.

The causes of uncertainties can be natural or man-made. The natural include environmental forces and actions caused by wind, hurricanes, tornadoes, snow, ice, temperature, water pressure, floods, and earthquakes. The natural variation can also result from construction or fabrication processes, of trying to stay within the required tolerances in preparation of materials (e.g. strength, modulus of elasticity), measuring the dimensions (width, length, depth, height).

Man-made uncertainties include forces and actions due to fires, vehicle and vessel collisions, changes of function (e.g. heavy storage area instead of office space), illegal overloads, poor maintenance, approximate methods of analysis and human errors.

The uncertainties can also be considered as aleatory and epistemic. Aleatory variability is the natural randomness in a process. For discrete variables, the randomness is parameterized by the probability of each possible value. For continuous variables, the randomness is parameterized by the probability density function. Epistemic uncertainty is the scientific uncertainty in the model of the process. It is due to limited data and knowledge. The epistemic uncertainty is characterized by alternative models. For discrete random variables, the epistemic uncertainty is modeled by alternative probability distributions. For continuous random variables, the epistemic uncertainty is modeled by alternative probability density functions. In addition, there is epistemic uncertainty in parameters that are not random but have only a single correct (but unknown) value.

In practice, the uncertainties are represented in terms variation of load and resistance parameters. These parameters are treated as random variables, described by cumulative distribution functions (CDF), or bias factor (ratio of the mean-to-nominal values), λ, and coefficient of variation, V (ratio of the standard deviation and the mean value). The new generation of bridge design codes is based on the reliability analysis performed using statistical models of load and resistance. The major steps in the development of the code include the selection of representative structures, formulation of limit states, development of load and resistance models, selection of the target reliability level, and finally the selection of load and resistance factors based on closeness to the target reliability. Reliability index, β, is an efficient measure of structural performance. The available methods for calculation of β are presented in the textbooks, e.g. [1]. The present paper deals with the statistical parameters of load and resistance.

2. Load models

The major load components include dead load, live load, environmental loads and special loads. Loads vary in time and the development of rational design provisions requires prediction of not only extreme load magnitude but also full load spectra, with the number of load cycles for the considered time period. The statistical parameters depend on function of the considered structure, exposure, age and degree of deterioration. They are often site-specific and point-specific even within a geographical location. Very important is the development of statistical parameters for load combinations or simultaneous occurrence of loads. The statistical parameters of load components available in literature are summarized below.

The statistical parameters for load components are taken from the available literature [2–4]. For each load combination, the statistical parameters are determined using the so-called Turkstra's rule [1, 5]. Turkstra observed that the extreme value of load combinations corresponds to the occur-

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rence of an extreme value of only one load component, while all the other load components take the corresponding average (arbitrary-point-in-time) values. Therefore, for each load component, two sets of statistical parameters are considered. The first corresponds to the maximum expected value of the load component during the life time of the structure, and the other corresponds to the average value (arbitrary-point-in-time value).

Dead load is the gravity load due to the self weight of the structural and non structural elements permanently connected to the bridge. Because of different degrees of variation, it is convenient to consider three components of dead load: weight of factory made elements (steel, precast concrete members), weight of cast-in-place concrete members, and weight of some special items for example of the wearing surface on bridges. All components of dead load can be treated as normal random variables. The bias factor (ratio of mean-to-nominal) value of dead load is, $\lambda = 1.05$, and coefficient of variation, $V = 0.10$ for cast-in-place concrete, and $\lambda = 1.03$, and coefficient of variation, $V = 0.08$ for steel and precast concrete [1]. Since dead load is assumed to be time-invariant, only one set of parameters is needed.

Live load is the gravity load due to weight of people, furniture, equipment, partitions and vehicles. It is strongly function-specific. There is a distinction between arbitrary-point-in-time and extreme live load. Assuming the design live load for office buildings is 250 kN/m^2, the arbitrary-point-in-time bias factor $\lambda = 0.24$ for small influence area (40 m^2) and 0.6 for large influence area (1000 m^2), and the coefficient of variation varies from 0.6–0.9 for small areas (40 m^2) to 0.2–0.4 for larger areas (1000 m^2), [2]. For the extreme live load, bias factor $\lambda = 1.0$ regardless of the influence area, and coefficient of variation varies as a function of the influence area, [2], from 0.15–0.25 for small areas (40 m^2) to 0.1–0.15 for larger areas (1000 m^2).

Statistical models of load and resistance for highway bridges are described by Nowak [4, 6, 7]. The main load combination includes dead load, live load and dynamic load. Live load includes the static and dynamic components. The static live load depends on many parameters including the span length, truck weight, axle loads, axle configuration, position of the vehicle on the bridge (transverse and longitudinal), number of vehicles on the bridge (multiple presence), traffic volume, girder spacing, and stiffness of structural members (slab and girders).

An extensive data base of weigh-in-motion (WIM) vehicles was processed by the research team at the University of Nebraska, including 65 million trucks [8–11]. Examples of the cumulative distribution function (CDF) of the gross vehicle weight (GVW) are shown in Fig. 1 on the normal probability paper. The curves correspond to data collected at various WIM locations in Florida. For comparison, the CDF of GVW obtained for vehicles on Ontario is also shown.

The WIM trucks were run over influence lines to obtain the maximum moments. Bias factors were calculated as the ratio of the WIM truck moment and the design moment. The design moment is calculated according to the AASHTO Code, 2011 [12]. Examples of CDF's of the bias factor are plotted in Fig. 2 for one of the WIM locations and spans from 12 to 60 m. There are considerable site-specific differences, and the heaviest vehicles were recorded in New York City. However, the number of extremely heavy trucks is very small compared to the total number of recorded trucks. Nevertheless, these extreme trucks have a significant effect on the upper tail of the CDF. For example, the removal of less than 0.1% of extreme vehicles changes the CDF as shown in Fig. 3. The solid bold curve in the full CDF and the thin curve corresponds to a truncated CDF.

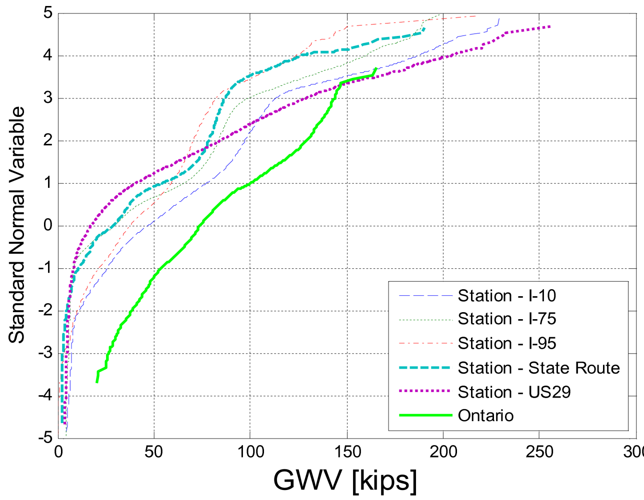

Fig. 1. CDF of GVW of vehicles recorded at locations in Florida

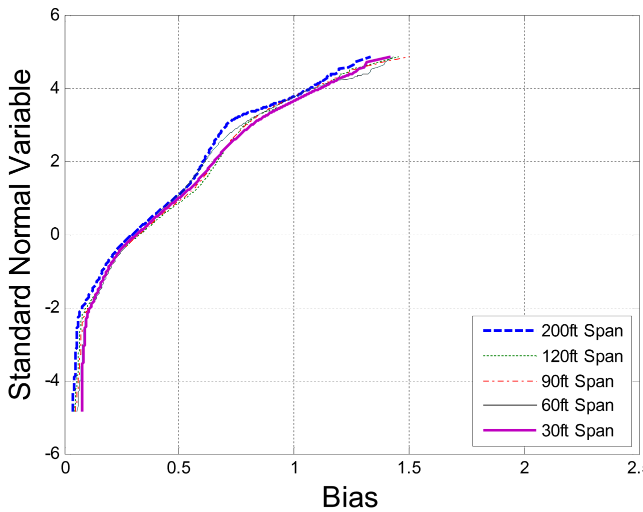

Fig. 2. CDF of bias of the mid-span moments for a location in Florida

In the United States, bridges are designed for economic life of 75 years. The available WIM data typically represents one year of traffic. Therefore, the results have to be extrapolated. For the average daily truck traffic (ADTT) of 5000, the bias factor for 75 year maximum moments is from 1.3 to 1.5, and coefficient of variation is 0.12.

The dynamic load model is a function of three major parameters: road surface roughness, bridge dynamics (frequency of vibration) and vehicle dynamics (suspension system). It was observed that dynamic deflection is almost constant and it does not depend on truck weight. Therefore, the dynamic load, as a fraction of live load, decreases for heavier trucks, and it does not exceed 0.15 of live load for a single truck and 0.10 of live load for two trucks side-by-side, [13].

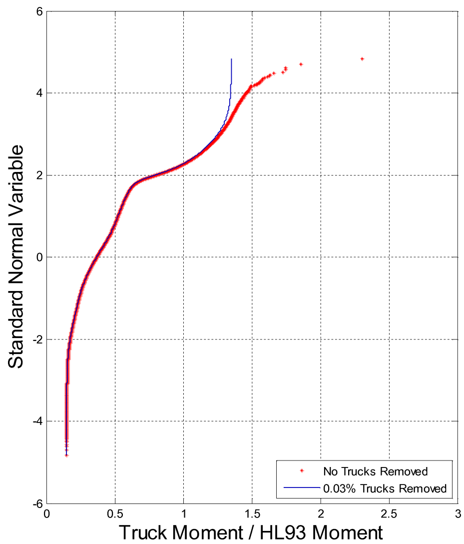

Fig. 3. Comparison of full CDF and truncated CDF by removal of less than 0.1% of extreme vehicles

Snow is specified in terms of ground snow, and the design value is modified using several additional parameters. The parameters for the maximum snow can be taken as $\lambda = 0.82$ and $V = 0.26$. For the average snow load, $\lambda = 0.20$ and $V = 0.87$.

Wind load is specified in terms of wind velocity at 10 m above the ground level, and the design value of pressure due to wind is modified using several additional parameters, including gust factor and exposure factor. The parameters for the maximum 50-year wind can be taken as $\lambda = 0.78$ and $V = 0.37$. It was assumed that the average (arbitrary-point-in-time) wind load is negligible, with $\lambda = 0$.

Earthquake load is specified in terms of ground acceleration, and the design value is modified using several additional parameters. The parameters for the maximum earthquake are $\lambda = 0.66$ and $V = 0.56$. For the average earthquake, $\lambda = 0$.

Table 1
Statistical Parameters for Load Components

Load Component	Arbitrary-Point-in-Time Load		Max 50 Year Load	
	Bias	COV	Bias	COV
Dead Load (cast-in-place)	1.05	0.10	1.05	0.10
Dead Load (plant cast)	1.03	0.08	1.03	0.08
Live Load (office buildings)	0.24	0.65	1.00	0.18
Snow	0.20	0.85	0.82	0.26
Wind	0.05	0.80	0.78	0.37
Earthquake	0.01	1.00	0.66	0.56

The load statistical parameters are summarized in Table 1. The parameters shown in Table 1 are bias factor and coefficient of variation. For each design case considered, the mean value of load is calculated as a product of the nominal (design) value and bias factor. The standard deviation is calculated as the product of the mean and coefficient of variation.

3. Resistance models

The capacity of a bridge depends on the resistance of its components and connections. The component resistance, R, is determined mostly by material strength and dimensions. R is a random variable. The causes of uncertainty can be put into three categories: (1) material factor including strength of material, modulus of elasticity, cracking stress, and chemical composition, (2) fabrication factor including geometry, dimensions, and section modulus, and (3) analysis factor including approximate method of analysis, idealized stress and strain distribution model. The resulting variation of resistance has been modeled by tests, observations of existing structures and by engineering judgment. The information is available for the basic structural materials and components. However, structural members are often made of several materials (composite members) which require special methods of analysis. Verification of the analytical model may be very expensive because of the large size of members. Therefore, the resistance models are developed using the available material test data and by numerical simulations.

The load carrying capacity or resistance, R is considered as a product of the nominal resistance, R_n and three parameters: strength of material, M, fabrication (dimensions) factor, F, and analysis (professional) factor, P,

$$R = R_n M F P, \tag{1}$$

the mean value of R, $\mu_R = R_n \mu_M \mu_F \mu_P$ and coefficient of variation, $V_R = (V_M^2 + V_F^2 + V_P^2)^{0.5}$, where, μ_M, μ_F, and μ_P are the means of M, F, and P, and V_M, V_F, and V_P are the coefficients of variation of M, F, and P, respectively. The statistical parameters are developed for steel girders, composite and non-composite, reinforced concrete T-beams, and prestressed concrete AASHTO-type girders [4].

The uncertainty in strength of materials can be assessed using the recent test data. An extensive data base was processed by the research team at the University of Nebraska [14]. For the compressive strength of ordinary concrete, f'_c, CDF's are shown in Fig. 4 on the normal probability paper. The bias factors vary from 1.3 for lower grade concrete (20.5 MPa) to 1.08 for high strength concrete (82.5 MPa), and coefficient of variation varies from 0.17 to 0.11. For light-weight concrete [15], CDF's of f'_c are plotted in Fig. 5, with bias factor, from 1.4 for lower grade concrete (20.5 MPa) to 1.16 for high strength concrete (49.0 MPa), and coefficient of variation varies from 0.16 to 0.12.

For reinforcing bars, CDF's of the yield stress are plotted in Fig. 6. The bias factor, $\lambda = 1.13$ and $V = 0.03$. This very low coefficient of variation is because all reinforcing steel in USA is made of recycled material. For prestressing strands, CDF's are shown in Fig. 7, with $\lambda = 1.04$ and $V = 0.02$. For structural steel shapes, the statistical parameters can be taken from [2]. The bias factor for yield stress is $\lambda = 1.10$ and $V = 0.10$.

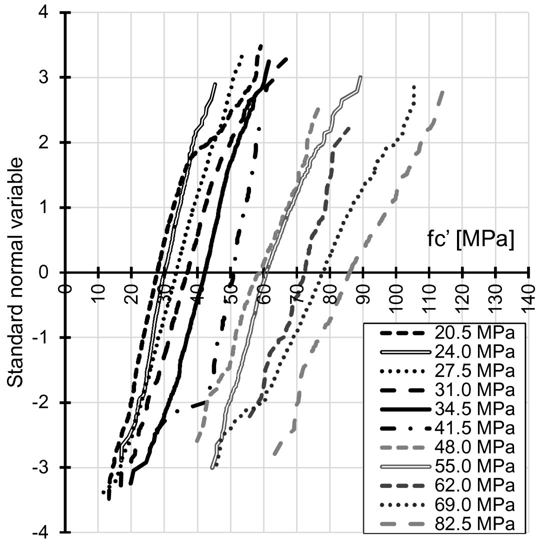

Fig. 4. CDFs of the compressive strength of ordinary and high strength concrete

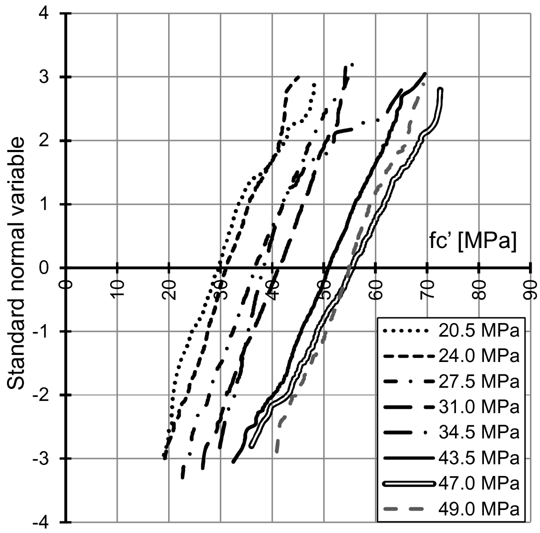

Fig. 5. CDFs of the compressive strength of lightweight concrete

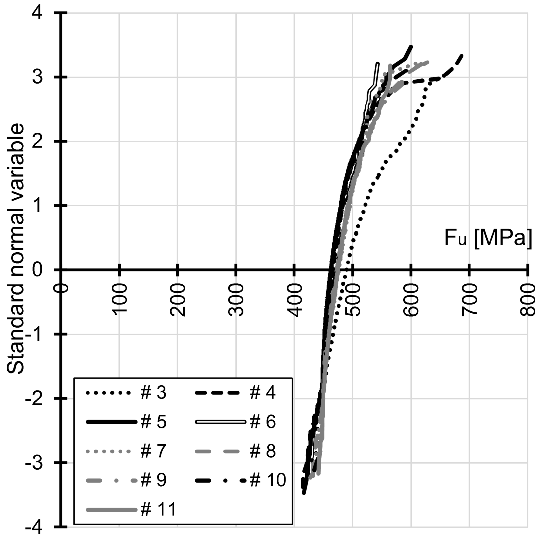

Fig. 6. CDFs for the tensile strength of rebars with bar diameters from 9 to 34 mm

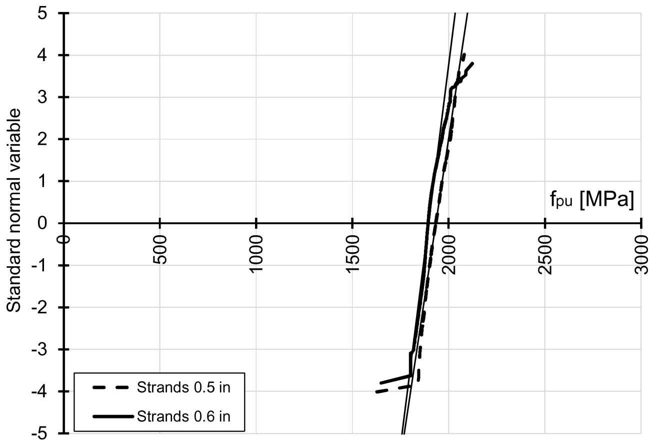

Fig. 7. CDFs of the tensile strength of prestressing strands with diameters of 12.5 mm and 15 mm

The statistical parameters of the fabrication and professional factors can be taken from [2]. They vary, and λ = 1.0-1.05 and V = 0.01-0.04 for dimensions and λ = 1.0-1.05 and V = 0.04-0.06 for professional factor.

The statistical parameters for beams were evaluated using Monte Carlo simulations. For steel girders, the parameters of R are λ_R = 1.12 and V_R = 0.10 for moment and λ_R = 1.14 and V_R = 0.105 for shear. For reinforced concrete T-beams, the parameters of R are λ_R = 1.12 and V_R = 0.135 for moment and λ_R = 1.20 and V_R = 0.155 for shear. For prestressed concrete, λ_R = 1.05 and V_R = 0.075 for moment and λ_R = 1.15 and V_R = 0.14 for shear.

4. Reliability analysis

The available reliability methods are presented in several publications, e.g. [1]. The reliability index is defined as a function of probability of failure, P_F,

$$\beta = -\Phi^{-1}(P_F), \tag{2}$$

where Φ^{-1} = inverse standard normal distribution function. There are various procedures available for calculation of β. In this study, β, is calculated using an iterative procedure and Monte Carlo simulations.

Two types of limit states are considered. Ultimate limit states (ULS) are mostly related to the bending capacity, shear capacity and stability. Serviceability limit states (SLS) are related to gradual deterioration, user's comfort or maintenance costs. The serviceability limit states such as fatigue, cracking, deflection or vibration, often govern the bridge design. The main concern is accumulation of damage caused by repeated applications of load (trucks). Therefore, the model must include the load magnitude and frequency of occurrence, rather than just load magnitude as is the case in the ultimate limit states. For example, in prestressed concrete girders, a crack opening under heavy live load is not a problem in itself. However, a repeated crack opening may allow penetration of moisture and corrosion of the prestressing steel. The critical factors are both magnitude and frequency of load. Other serviceability limit states, vibrations or deflections, are related to bridge user's comfort rather than structural integrity.

The reliability analysis procedures can be used for a comparison of different variants of design alternatives, materials and types of structure. Optimum safety level can also be expressed in terms of the target reliability index. The development of load and resistance factor design (LRFD) codes for the design of bridges requires the knowledge of the target reliability level. The optimum safety level depends on the consequences of failure and cost of safety [16]. Selection of the target value can be based on consideration of these two parameters. Target reliability indices calculated for newly designed bridges and existing structures are different for many reasons. Reference time period is different for newly designed and existing bridges. New structures are designed for 50–75 year life time and existing bridges are checked for 5 or 10 year periods. Load model, used to calculate reliability index depends on the reference time period. Maximum moments and shears are smaller for 5 or 10 year periods than for 50–75 year life time. However, the coefficient of variation is larger for shorter periods. Single load path components require a different treatment than multiple load path components. In new designs, single load path components are avoided, but such components can be found in some existing bridges. Target reliability index is higher for single load path components.

Reliability indices calculated for existing bridges can be considered as the lower bounds of safety levels acceptable by the society. A drastic departure from these acceptable limits should be based on an economic analysis. The target reliability index depends on costs and has different value for a newly designed bridge and an existing one. In general, it is less expensive to provide an increased safety level in a newly designed structure. For bridges evaluated for 5 or 10 year periods (intervals between inspections), it is assumed that inspections help to reduce the uncertainty about the resistance and load parameters. Therefore, the reliability index can be lower for existing bridges evaluated for 5 or 10 year periods. Because of economical reasons, it is convenient to differentiate between primary and secondary components in bridges. The difference between these components depends on the consequences of failure. Target reliability index for secondary components is lower than that for primary components.

The analysis is performed for the ultimate limit states (ULS) and serviceability limit states (SLS). Serviceability limit states have a lower level of consequences of failure. Therefore, lower values of the target reliability index are selected for SLS than ULS. For the ultimate limit states, calculated reliability indices represent component reliability rather than system reliability. The reliability indices calculated for structural system are larger than for individual components by about 2. Therefore, selection of the target reliability level should be based on consideration of the system. Then, target reliability index for components can be derived using the appropriate formulas. For serviceability limit states, reliability indices vary considerably depending on the limit state. For example, the consequences of exceeding the tension stress limit in concrete girders are much less severe compared to the ULS.

Recommended values of the target reliability indices for design and evaluation of bridges are listed in Table 2. The numbers are rounded off to the nearest 0.25. For SLS in prestressed concrete girders, the compression stress limit is considered to prevent the formation of an excessive permanent deformation (kink) in the girder. The consequences of exceeding the tension stress limit are much less severe compared to the ULS. Therefore, the proposed target reliability index for tension is $\beta_T = 1.0$ [17]. For compression stress, the target reliability is $\beta_T = 3.0$.

Table 2
Recommended target reliability indices for design and evaluation, ULS

Time Period	Primary Components Single Path	Multiple Path	Secondary Components
5 years	3.50	3.00	2.25
10 years	3.75	3.25	2.50
50 years	4.00	3.50	2.75

5. Human error

A major source of uncertainty in the building process is human error. It is defined as a departure from acceptable practice [18]. There are three types of errors: conceptual, intentional and execution as shown in the flowchart, Fig. 8. Surveys show that over 90% of structural failures are due to human error, with about 50% in the design and 50% in construction. Therefore, in practice, risk mitigation requires control of errors. The error control can be approached from either reduction/prevention of occurrence or reduction of consequences. The first one involves checking procedures, inspections and psychological considerations (motivation, working conditions), fool proof design, and so on. Control of error consequences requires identification of the most sensitive components, connections and/or construction procedures, and providing adequate safety reserve accordingly. The latter can be accomplished by sensitivity analysis [18].

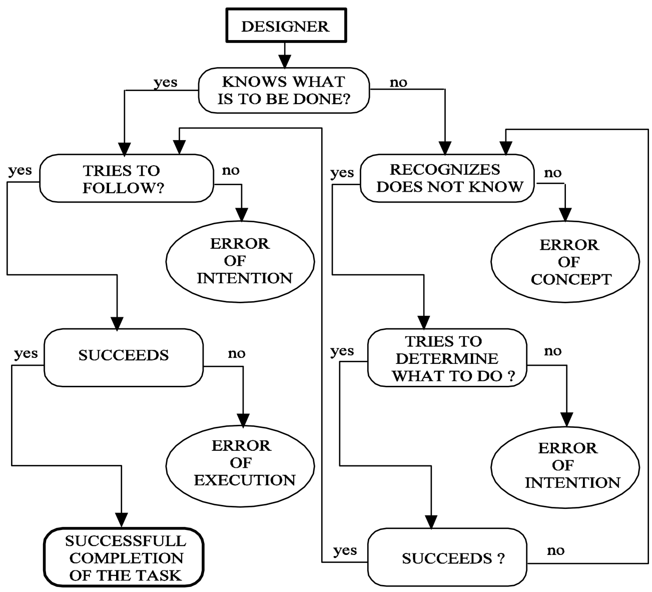

Fig. 8. Flowchart of the design process and human errors

A simple example of a sensitivity analysis is consideration of simply supported beams, designed for a uniformly distributed load as shown in Fig. 9a. There are three different materials considered: steel, prestressed concrete and wood. The coefficients of variation of resistance are 10.5%, 6.5% and 22.5%, respectively. It is assumed that the uniformly distributed load is a random variable and three cases of the coefficient of variation were considered: 8%, 12% and 15%. For each combination of material and coefficient of variation of load, a beam is designed so that the reliability index is 3.5. Then, human errors are considered in form of distribution of the load over a reduced portion of the span. The ratio of the loaded span and total span is denoted by α (Fig. 9b). Factor α can take values from 1.0 corresponding to fully distributed load (Fig. 9a) to 0 corresponding to all the load concentrated at mid-span (Fig. 9c). The reliability indices are calculated each value of α. The results are shown in Fig. 10 for a steel beam, Fig. 11 for prestressed concrete girder, and Fig. 12 for wood stringer. For a steel beam, the relationship between β and α is almost linear. For a prestressed concrete beam, even a small reduction of α caused a drastic drop in β. On the other hand, the reliability of a wood beam shows a very weak relationship with α. This is because the reliability calculation is dominated by the uncertainty in mechanical properties of material.

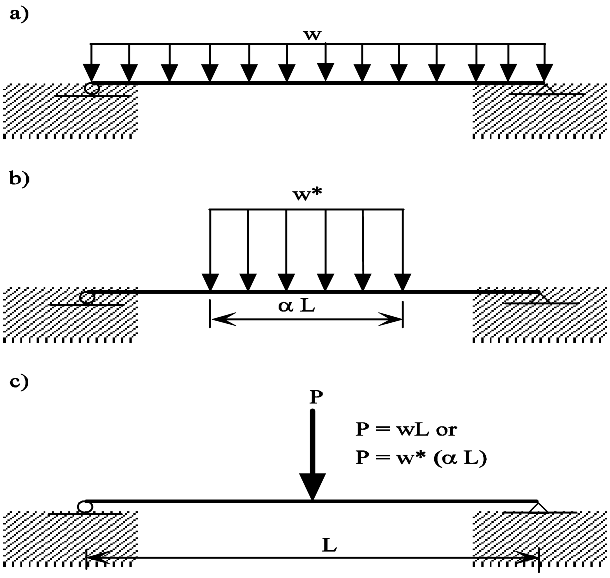

Fig. 9. Considered simply supported beams

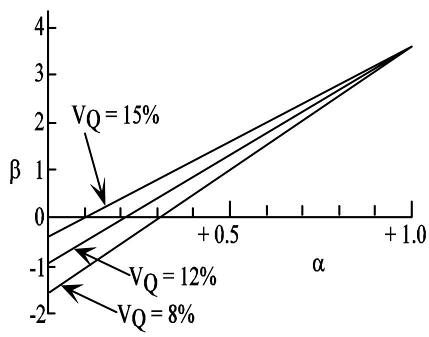

Fig. 10. Steel beam, $V_R = 10.5\%$ (flexural capacity of a compact section)

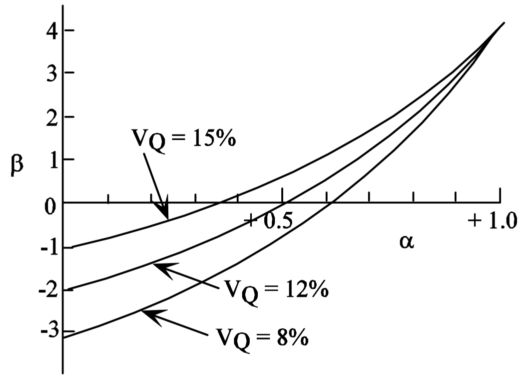

Fig. 11. Prestressed concrete bridge girder, $V_R = 6.5\%$

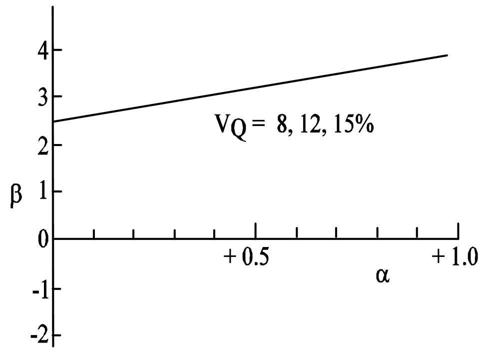

Fig. 12. Timber stringer, Douglas-fir, 150 $\times$ 400 mm, $V_R = 22.5\%$

The example of the load erroneously distributed over a reduced portion of the span (Fig. 12) may appear as a trivial case; however, this is what happened recently with fatal consequences. The I-35W Bridge over Mississippi in Minneapolis suddenly collapsed during the afternoon rush hour on August 1, 2007, killing 13 people and injuring 145. The bridge failure was attributed to a design error that underspecified the thickness of steel gusset plates connecting the truss members at a joint. However, this was not the only reason which leads to failure. The bridge was undergoing repairs and the construction equipment and materials were placed in the mid-span over a small area. This concentrated load was the main reason for overloading under-sized gusset plates and the progressive collapse of the entire bridge.

6. Conclusions

The uncertainties in the building process are due to natural variation of environmental loads (wind, snow, ice, earthquake, temperature, flood, hurricane, and tornado), natural variation of material properties, and man-made causes and human errors in particular. Their effect can be quantified and expressed in terms of statistical parameters such as bias factor and coefficient of variation. Statistical parameters are presented for materials and loads. It has been observed that the quality of materials has improved over the years as indicated by reduced coefficients of variation. Typical values of reliability indices are shown. The effect of human error on reliability can be established using the sensitivity analysis and sensitivity functions.

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