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THEME 2

DESIGN CONCEPTS AND METHODS

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Serviceability Criteria For Building Codes

Critères d'aptitude au service dans les règlements de construction

Gebrauchstauglichkeitskriterien für Bauwerksnormen

Robert H. LEICESTER
Chief Res. Scientist
CSIRO
Melbourne, Australia

Dr. Leicester undertakes research related to timber engineering, structural reliability and the impact of natural hazards on buildings. He has contributed to several Australian and international building Standards.

SUMMARY

The paper presents simple statistical models for processing data for design codes and performance standards. A literature review of the relevant data is made for the cases of serviceability limits related to building deformation, sway, floor vibration and cracking. It is found that the impact of unserviceability parameters on humans is highly variable and is influenced by many non-structural parameters.

RESUME

L'auteur présente divers modèles statiques simples, en vue de traiter les données relatives aux règlements de dimensionnement et aux normes de qualité. A partir de l'étude de publications, il effectue un choix de données essentielles relatives aux états limites de la déformation des ouvrages, du déplacement horizontal des étages, de la vibration des planchers et de la fissuration. Il en résulte que les paramètres d'inaptitude au service ont un effet fort variable sur les hommes et qu'ils sont influencés par de nombreux facteurs non structuraux.

ZUSAMMENFASSUNG

Der Beitrag stellt einfache statische Modelle vor, um Daten für Bemessungsnormen und Güterichtlinien zu bearbeiten. In einer Literaturstudie wird das betreffende Datenmaterial bezüglich Grenzzuständen der Bauwerksverformung, Stockwerksverschiebung, Deckenschwingung und Rissbildung gesichtet. Wie sich herausstellt, ist die Wirkung von Kenngrössen unzulänglichen Gebrauchsverhaltens auf Menschen sehr unterschiedlich und von vielen nicht-baulichen Einflüssen bestimmt.

1. INTRODUCTION

1.1 Serviceability

Because of the increased sophistication in our knowledge on structural strength and the use of higher strength and lighter weight materials, serviceability considerations have become a prime consideration in structural designs. Some idea of this transition can be obtained by noting that whereas the sway in a strong wind of the Empire State building is about 100 mm, the sway of modern skyscrapers such as the World Trade Centre in New York may be as high as 1000 mm.

In this paper, the term serviceability will be taken to refer to all structural behaviour, excluding structural collapse, that renders a building or construction unfit for its intended use. This lack of fitness may relate to human reactions (aesthetic, physiological or psychological), and may range from annoyance to medical trauma; it may also relate to matters that hinder the operations of humans or equipment; it does not include matters related to collapse due to corrosion or fatigue. In concept at least, it is possible to modify an unserviceable building, so that it becomes serviceable.

Some excellent summaries of the state-of-the-art with respect to design for serviceability limit states are to be found in the 1988 symposium/workshop held at Ottawa [2], the report by the ASCE ad hoc committee [11] and the BRANZ study report [10]. Other useful summaries on specific aspects include studies related to deformations [16,20,52], vibration loads [4,5,21,22,23,24,33,50], floor vibrations [5,13,18,35] and cracking [31].

1.2 Codification

The evolution of a structural technology can be divided into three phases as follows. In the first phase, the structural engineering is undertaken successfully only by expert engineers, operating largely through a mixture of past experience and intuition; in this phase, engineering may be considered to be an 'art'. In the second phase, limited research is undertaken to provide these master engineers with information that will assist them in pursuing their art. Eventually a third phase occurs when there is enough information and experience to enable the derivation of design procedures through formal processing of the available data; where possible, this is the preferred option for use in the drafting of codes and standards.

The use of codes and standards within the building industry has been discussed at length in a previous paper [28]. In particular these documents play a role as part of a formal agreement between two or more parties; they define their relative duties and responsibilities in the design and production of a building. Codes and Standards are also useful in that they provide a framework for the collation of data both from research and feedback from field experiences. In addition, it should be noted that codification of design procedures enables engineers of modest ability to execute competent designs of conventional structures.

In the following two simple statistical models of the codification process are presented. They are used as a framework for examining the suitability of available data for the derivation of design criteria related to the specific cases of building deformation, building sway, floor vibration and element cracking.

2. MODELS FOR USE IN CODIFICATION

2.1 General

Because serviceability involves human actions and response, it is a complex matter, involving high variability and nonlinear functions. In the following it will be assumed that for codification purposes, serviceability criteria will take the form of a finite set of simple design decisions. Each such decision will involve an effective cost, and the code recommendation will be based on minimising this cost.

Two types of codes or standards will be considered. The first will be a design code; the second will be an in-service performance standard.

2.2 Design Code

In concept, this will be a design code that is optimised from the viewpoint of the building owner. A statistical model, discussed in a previous paper [29], is used to develop this code and is illustrated schematically in Figure 1; it is stated in terms of an unserviceability parameter such as crack width. The scenario assumed is that a building has an in-service value T of the unserviceability parameter; should this parameter exceed the value of the tolerance level or complaint threshold of the client, denoted by U, then an effective additional cost C_F will be incurred. This cost may be taken to include not only direct costs, such as remedial costs, but also indirect costs that may arise, for example, from bad publicity or the loss of tenants. The aim is to optimise the value of $\bar{T}/\bar{U}$ chosen for the design procedure.

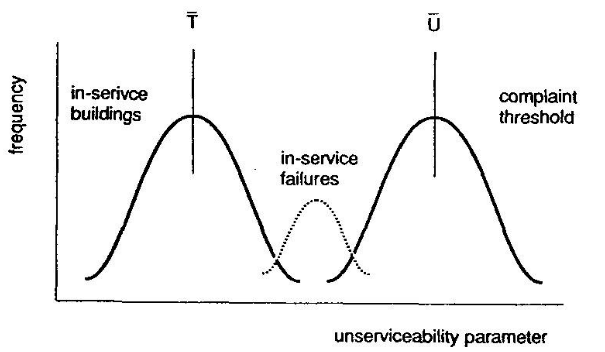


Fig. 1 Statistical model for a design code.

For the building owner, the cost associated with the design denoted by C, can be written

$$C = C_S + C_F\, p_F \tag{1}$$

where C_S denotes the cost of the structure and $p_F = \Pr(U < T)$

If it is assumed [29] that

$$C_S = A\bar{T}^{-m} \tag{2}$$

and

$$p_F = B(\bar{U}/\bar{T})^{-n} \tag{3}$$

where A, B, m and n are constant, then optimisation of equation (1) leads to

$$(\bar{T}/\bar{U}) = [n\, B\, C_{FO}/m]^{1/n} \tag{4}$$

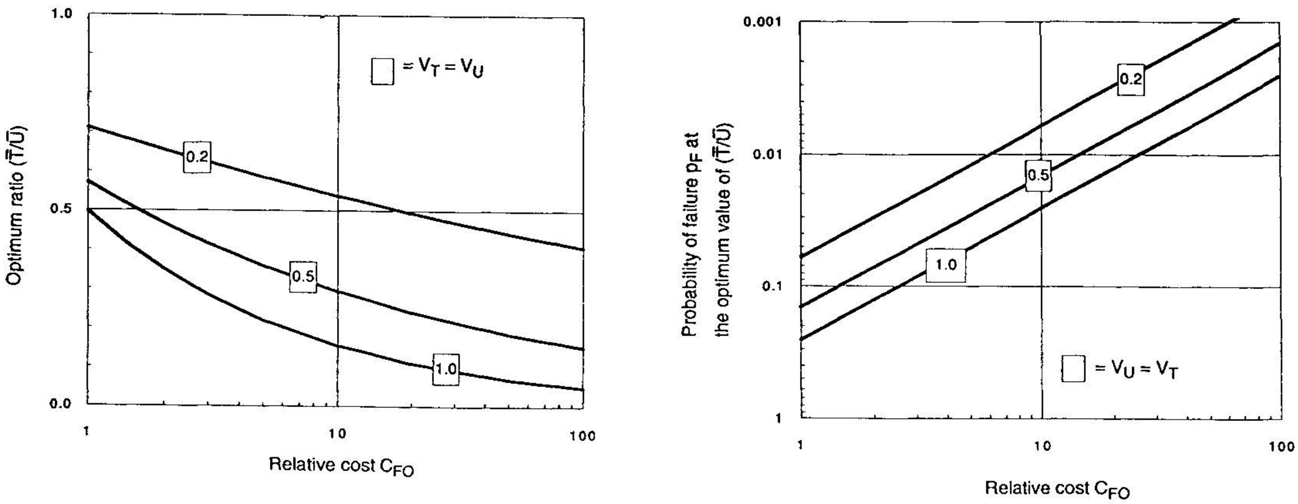
and

$$p_F = m/n\, C_{FO} \tag{5}$$

where

$C_{FO} = C_F/C_{SO}$, and C_{SO} denotes the cost of the optimum structure.

Appendix A gives a method for estimating the parameters B and n for use in equations (4) and (5); these parameters are stated in terms of V_T and V_U, the coefficients of variation of T and U respectively. Some typical optimised values of $\bar{T}/\bar{U}$ and p_F based on these assumptions are shown in Figure 2.



(i) Ratio ($\bar{T}/\bar{U}$) (ii) Probability of failure

Fig. 2 Optimum values for design codes (m = 0.5).

2.3 Performance Standards

The statistical model for this case is illustrated schematically in Figure 3. Here an in-service value L of the unserviceability parameter is specified as a legal limit. If this limit is exceeded, the builder must pay a remedial cost C_F. If the limit is not exceeded, but the unserviceability parameter exceeds the complaint threshold of the owner, then the owner will pay for the costs of remedial action.

The cost to the building owner is

$$C = C_S + C_F \, p_{F1} \tag{6}$$

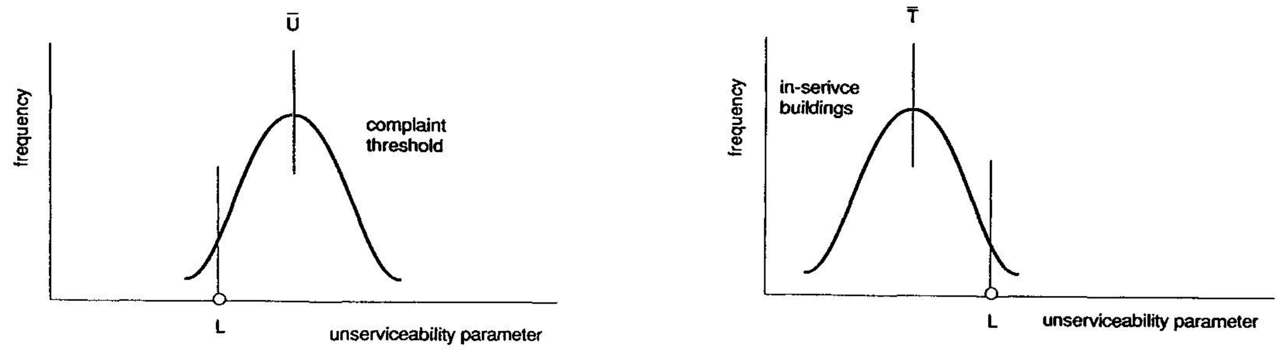
where $C_S = A_1 L^{-m}$, A_1 is a constant and $p_{F1} = Pr(U < L)$.

The cost to the builder is

$$C = C_S + C_F \, p_{F2} \tag{7}$$

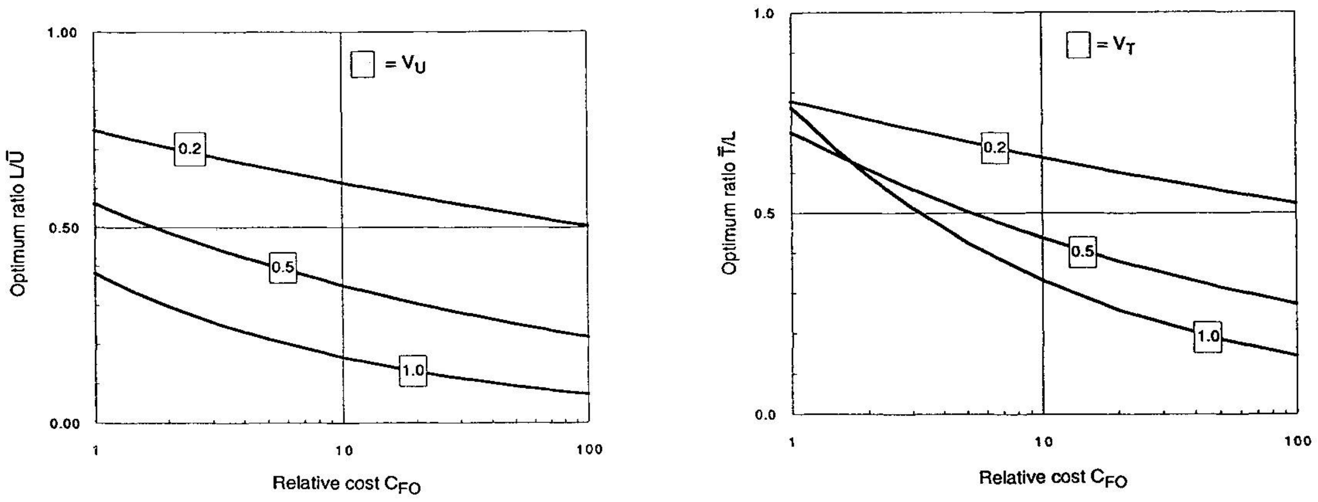
where $C_S = A_2 L^{-m}$, A_2 is a constant and $p_{F2} = Pr(T > L)$.

It is now assumed that first the building owner selects the legal limit L so as to minimise his costs, and then the builder selects the target in-service value of $\bar{T}$ so as to minimise his costs. Then the optimisation of equations (6) and (7) leads to equations identical to those for the optimisation of equation (1), except that the coefficients of variation $V_T = 0$ and $V_U = 0$ are to be used in the optimisation of equations (6) and (7) respectively.
Some optimum solutions for these cases are shown in Figure 4.



(i) Selection of a legal limit L (ii) Builder response

Fig. 3 Statistical model for a performance standard.



(i) Selection of a legal limit (ii) Builder response

Fig. 4 Optimum values for performance standards (m = 0.5)

3. BUILDING DEFORMATIONS

3.1 Unserviceability Parameters

Unserviceability parameters for building deformations include deviations from straight lines, distorted right angles, tilt of walls and slopes of floors [20,46].

3.2 Human Response

The impact of these parameters is considerably influenced by additional architectural parameters such as the incident angle of surface lighting, the surface colour and texture, and whether there are any visual references, such as a free-standing cupboard next to a wall, to assess the magnitude of the deviations [44,46].

The writer is unaware of any direct measurements of the statistical characteristics of human response to the above unserviceability parameters.

3.3 In-service Values

With respect to in-service values of the unserviceability parameter, there is an interesting study by Espion and Halleux on the long-term deflection of reinforced concrete beams [14]. They observed a coefficient of variation of 35 per cent in the ratio of actual deformation to predictions by ACI and CEB formulae; this variability is a measure of V_T.

4. BUILDING SWAY

4.1 Unserviceability Parameter

Probably the most common choice for the unserviceability parameter is linear acceleration [32].

4.2 Human Response

When a building sways excessively, humans become aware of linear accelerations, angular accelerations, jerks (rates of change in acceleration), visual stimuli and sound stimuli [7,19,51]. The response ranges all the way from 'feeling refreshed' to nausea to acrophobia.

Figure 5 shows the results of laboratory studies on human perception to horizontal motion undertaken by Chen and Robertson in 1972 [8]; the results indicate a coefficient of variation of about 50 per cent in the perception threshold of horizontal accelerations; there is also an additional factor of 2, depending on whether the subject was seated or standing. This variability is a measure of V_U. Field surveys by Hansen and Reed [19] and by Takeshi Goto [51] in the aftermath of major wind storms has revealed a wide scatter between people with respect to the frequency that is considered to be acceptable for experiencing specific wind storms.

4.3 In-service Values

With regard to in-service performance, a survey of building measurements by Ellis has shown that there is a coefficient of variation of about 30 per cent in the uncertainty associated with predicting the fundamental frequency of vibration of a building [12]. Comments by Jeary indicate that the coefficient of variation of the error associated with prediction of building response is likely to be as high as 100 per cent [25]. These variabilities are indicative of the magnitude of V_T.

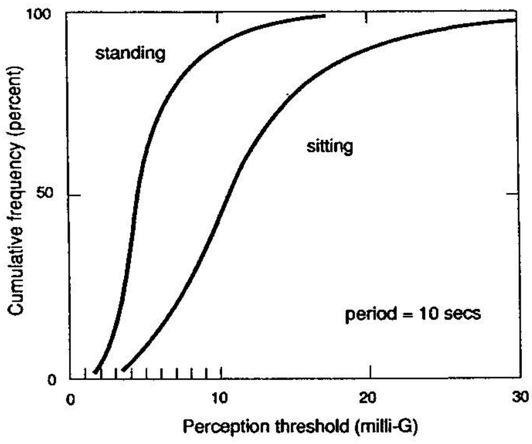


Fig. 5 Perception threshold for horizontal vibrations, after Chen and Robertson [8].

5. FLOOR VIBRATIONS

5.1 Unserviceability Parameters

Choices for unserviceability parameters related to human response to vibrations have included numerous complex functions of displacement, velocity, acceleration, frequency and damping [5]. Murray has compared the GSA, CSA, ISO and modified Reiher-Meister scales for this purpose and found significant discrepancies between them [33].

Even within the narrow topic of wooden floors there is a variety of choices for the unserviceability parameter. For floors with a natural frequency above 8 Hz, Ohlsson uses the peak velocity arising from a 1 N–s impulse [36]; Chui and Smith use the peak acceleration due to a heel-drop loading [9]; Onysko and Russell both use the deflection due to a static load [37,45]. In addition, Smith and Chui make a suggestion that in practice the vibration characteristics of a light weight floor are more likely to be dominated by the disposition of the superimposed loading rather than by the structural characteristics of the floor itself [47].

5.2 Human Response

Some idea of the variability of human response to floor vibrations can be obtained from the studies on 40 persons by Wiss and Parmlee in 1974 [55]. A coefficient of variation of about 30 per cent was obtained both for the threshold value and the strongly perceptible value of the frequency x displacement parameter of transient vertical vibrations.

Another estimate of variability is given in the 1954 research paper by Russell illustrated in Figure 6 [45]. In his study the unserviceability parameter was taken to be the midspan deflection of wood floor systems when subjected to a 1.7 kN midspan point load. For the 225 persons involved, the deflection associated with acceptance involved a coefficient of variation of 35 per cent. Both Russell and Onysko have noted the significance of sound stimuli in the acceptance of a floor system [38,45].

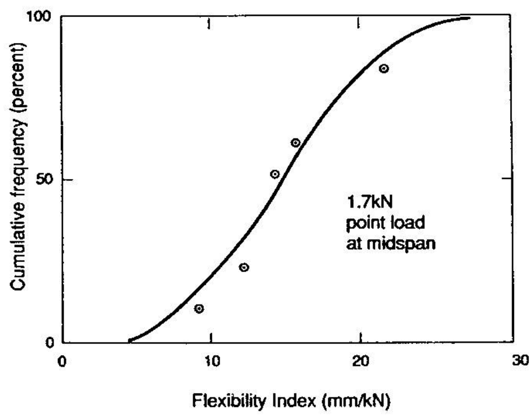


Fig. 6 Rejection threshold for wooden floors,

5.3 In-service Values

For simple floor systems, the variability of deflection and vibration characteristics can be estimated quite accurately from the variability of the materials used. However, there are difficulties. For example, with wooden floors the effects of gaps in the sheeting material and the complex nature of damping introduce many uncertainties [41].

In a study on long span floor systems, Allen and Rainer observed a coefficient of variation of 30 per cent in the ratio between the measured and calculated accelerations due to heel impacts on floor systems [3]; these accelerations are a popular choice for the unserviceability parameter of long span floor systems.

6. CRACKING

6.1 Unserviceability Parameter

The most usual parameter for unserviceability is crack width, although crack length and the number of cracks per unit area have also been considered.

6.2 Human Response

The impression of a crack is considerably influenced by secondary parameters such as the mode of lighting, the surface texture, the occurrence of dirt within the cracks and the viewing distance [6]. Apart from aesthetics, there appears to be a strong psychological element in the human response to cracks. For example, in their survey on human response, Padilla and Robles used attitude scales with questions containing the phrases 'give a bad impression', 'annoy me', 'proof that bad materials were used', 'feeling of danger' and 'fear that the apartment will collapse' [39].

Figure 7 shows data from a study by Haldane involving 400 persons asked to assess cracks in a simulated stub column [17]. The coefficient of variation of crack width corresponding to the rejection limit is about 40 per cent. Some data on field observations of the complaint threshold for cracks in brickwork has been given by Walsh [54].

6.3 In-service Values

In a study by Prakash and Desayi, the ratio of measured to computed crack widths in reinforced concrete beams, slabs and tension members was found to have a coefficient of variation of about 30 per cent. These were cracks due to static loads.

In their monumental study in 1970, Mayer and Rusch measured a coefficient of variation of about 30 per cent in the prediction of building damage due to the deflection of reinforced concrete building components [31].

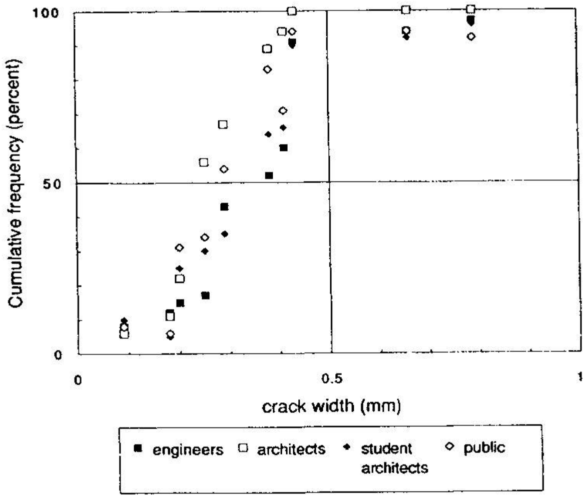


Fig. 7 Rejection threshold for crack widths in reinforced concrete stub columns, after Haldane [17].

Studies related to the prediction of building cracks due to vibrations caused by construction machinery indicate an uncertainty corresponding to a coefficient of variation of 30–60 per cent [15,34].

7. COSTS

The costs associated with resisting unserviceability are easily obtained for any specific theory of resistance. For example, the mass per unit length of Australian universal beams is proportional to $I^{0.414}$, where I denotes the second moment of area [30]. Hence, in the use of these beams, m = 0.414 in equation (2) when the unserviceability parameter is related to beam flexibility.

The costs associated with remedial action are not readily available. Some estimates have been given in a previous paper [29]. Interesting examples of costs related to cracking has been published by Kitcher and by Reid and Turkstra [27,43].

8. CODIFICATION

8.1 Data Processing

From this study and a previous one, typical values of parameters for the statistical model are $V_T = 0.2 - 1.0$, $V_U = 0.2 - 1.0$, $m = 0.2 - 1.0$ and $C_{FO} = 1 - 20$ [29].

For most cases a high probability of failure is associated with serviceability limit states, and so the shape and tails of the distributions are not as critical as in the case of analyses of ultimate limit states. Furthermore, if the parameter T turns out to be a complex function of several variables, then simple first order approximations may be used to derive acceptable values of $\bar{T}$ and V_T.

When data is very limited, the coefficients of variation V_T and V_U can be estimated from studies on similar phenomena, the ratio $\bar{T}/\bar{U}$ obtained from the statistical model, and then the mean value of $\bar{T}$ or $\bar{U}$ chosen so as to provide a match for any available data on either successful or unsuccessful inservice structural behaviour as indicated in Figure 1.

8.2 Load Combinations

Load combinations for ultimate limit states are typically estimates of peak loads in a 50-year period; as such they are usually too extreme for use in checking many types of serviceability limit states. For example, the acceptable lateral sway of a building may be stated in terms of events per year [32].

Examples of alternative load combinations for use in checking serviceability limit states are given in the Australian Standard AS 1170.1 [49]; based on the work of Pham and Dayeh, these include the peak load in any one-year period and the mean sustained load, both having a five per cent chance of exceedance [40]; the mean sustained load is intended to be used in creep and settlement estimates.

9 OPERATIONAL CONSIDERATIONS

9.1 Data

The literature search has revealed that for purposes of formal processing, the available data is limited, even with respect to the modest requirements of the statistical models discussed in this paper. Data on the complaint threshold for real buildings is very meagre. However, the use of formal models, as an alternative to simply following the recommendations of master engineers, has several advantages. One reason is the fact that intuitive or heuristic processing of limited statistical data is known to be associated with major bias effects [26]. More importantly perhaps, is that the use of models leads to an awareness of the deficiencies in the existing data bank and provides some idea of the potential benefits to be gained from gathering further data.

One firm conclusion derived from the literature search is that the uncertainties T and U involve high variabilities, and that the optimum failure rates for serviceability limits are high in comparison with that of ultimate limit states.

9.2 Design Codes

It is important that the intent of a design code be transparent; for example, major difficulties frequently arise in the application of many modern codes because it is not clear whether the purpose of deformation limits given therein are related to aesthetic or damage considerations. Ideally, a total scenario should be provided; it should include a description of the relevant failure mode and the associated range of remedial actions. In this regard it is interesting to note that the Australian Standard AS 2870 for residential footings on expansive soils, based on the work of Walsh, includes not only a description of the normal cracking to be expected, but also the reasonable care that the building owner is expected to take in the protection of these footings [48,54].

9.3 Performance Standards

If lengthy litigations on failures are to be avoided, then a critical aspect of performance standards is that they specify performance in terms of parameters that can be easily measured in the event of a dispute. Thus the crack width would be considered to be a useful parameter, whereas the lateral sway of a building in a 50-year return wind would not.

9.4 Multiple Limits

The literature review on the impact of unserviceability parameters on humans has revealed that these are strongly influenced by many nonstructural matters such as architectural features, audible and visual stimuli, building usage and the disposition of people. Thus, a strong case can be made that serviceability limits for both design codes and performance standards should not be specified as single values but rather should be specified in terms of sets of limits; this will permit the designer or building owner to choose limits that can be matched to each particular situation, and to the choice of building quality.

10. CONCLUSIONS

Simple statistical models have been presented for a design code and a performance standard. A literature review indicates that even for these simple models the data currently available for formal processing within the framework of these standards is limited. However, these models will become increasingly useful as data accumulates.

The human response to unserviceability parameters is found to be highly variable and to be influenced by many nonstructural parameters. Accordingly, a case can be made that serviceability limit states should be presented not as single values but as sets of values from which choices can be made to suit specific design situations.

Ideally, design codes should be transparent with respect to the failure scenario addressed by the design procedures; similarly performance standards should use criteria that are easily checked in the event of a dispute as to whether or not a serviceability limit state has been violated. The performance standard is probably the best option for countries with intense litigation practices.

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APPENDIX A
PROBABILITY OF FAILURE

Because relatively high probabilities of failure are involved in serviceability limit states, the choice of statistical distributions for the variables T and U is not critical. For convenience, log normal distributions will be chosen. The probability of failure is then given by

$$\begin{aligned} p_F &= \Pr(U < T) \\ &= \Phi(-\beta) \end{aligned} \tag{A1}$$

where

$$\beta = \frac{\ell n(\bar{U}/\bar{T}) + \ell n[(1 + V_T^2)/(1 + V_U^2)]^{1/2}}{\{\ell n[(1 + V_T^2)\,(1 + V_U^2)]\}^{1/2}} \tag{A2}$$

in which $\Phi(\ )$ denotes the cumulative distribution function of a unit normal variate.

To a reasonable approximation, equation (A1) may be written

$$p_F \approx 10^{-\beta} \tag{A3}$$

Equations (A2) and (A3) then lead to

$$p_F \approx B(\bar{U}/\bar{T})^{-n} \tag{A4}$$

where

$$n = 2.3/\{\ell n[(1 + V_T^2)\,(1 + V_U^2)]\}^{1/2} \tag{A5}$$

and

$$B = [(1 + V_T^2)/(1 + V_U^2)]\}^{-n/2} \tag{A6}$$

For coefficients of variation less than 0.3, $n \approx 2.3/\sqrt{V_T^2 + V_U^2}$ and $B \approx 1$.

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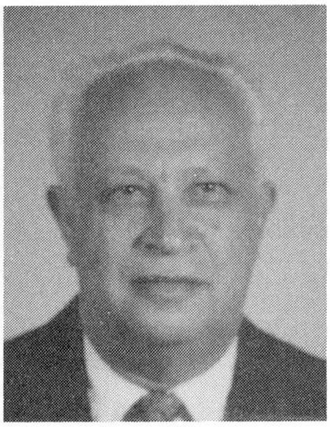
Structural Serviceability Depending on Multiple Parameters

Aptitude au service de constructions dépendant de plusieurs paramètres

Gebrauchstauglichkeit in Abhängigkeit von mehrfachen Parametern

Fiodor BLJUGER
Sen. Res. Associate
Technion
Haifa, Israel



F. Eph. Bljuger, born 1929, has been Researcher and Adjunct Lecturer at the Faculty of Civil Engineering, Technion since 1974. Prior to this he was Senior Scientist and Head of Laboratory at Moscow Institutes, since 1963. He is the author of many articles on reinforced concrete and of the book «Design and Precast Concrete Structures».

SUMMARY

A probabilistic cracking analysis of prestressed slabs based on an experimental model and on the variation characteristics of all essential parameters, is presented. Traditional estimations of serviceability in terms of cracking yields significantly different reliabilities for structures with different variable parameters. The situation may be improved with the aid of multiple analysis, whereby some of parameters can be taken as constants.

RESUME

Une analyse probabilistique de la fissuration des dalles précontraintes basée sur un modèle expérimental est mise en évidence; elle tient compte de la variation des caractéristiques de tous les paramètres essentiels. L'approche traditionnelle de l'aptitude au service basée sur la fissuration de constructions avec plusieurs paramètres variables donne des sécurités différentes. Cette situation peut et doit être améliorée à l'aide d'une analyse de plusieurs paramètres, certains paramètres pouvant être considérés comme constants.

ZUSAMMENFASSUNG

Aufgrund von Experimenten und der Variationseigenschaften aller wichtigen Parameter wird eine probabilistische Berechnung der Rissbildung in vorgespannten Platten entwickelt. Der herkömmliche Nachweis der Rissebeschränkung ergibt deutlich abweichende Tragwerkszuverlässigkeiten mit anderen variabeln Parametern. Der Unterschied verringert sich durch Einsatz mehrfacher Parameteranalyse, wobei einige Parameter konstant gehalten werden können.

1. INTRODUCTION

For many concrete structures, mainly prestressed members, the deciding factor has become serviceability in terms of cracking. A a rule, crack appearance depends on one, two or more variable parameters, such as loads, tensile strength of the concrete, residual prestress, topping weight and its shrinkage, etc.
The universally accepted method of verification of structure serviceability in terms of cracking is based on the characteristic values of most of the above-mentioned variable parameters [1-4]. It is known that structures differ from one another by the number of variable parameters, as well as by the degree of their variation. Consequently, different structures should have considerably differing service reliability [5-8]. At the same time not all variable parameters have a pronounced effect on the serviceability of a structure.
In this paper structural serviceability is analysed under allowance for all essential variable parameters: tensile strenth of the concrete (including its partial variation over the length of each member), prestress at the extreme fibres, weight of topping, shrinkage effects, as well as live loads. The cracking probability analysis, proposed by the author, is based on consideration of a structural model with given spacing of potential crack sites, predetermined by experiments, and on evaluation of the overall cracking probability of the considered structure as function of the above multiple parameters. It is shown that only part of the variable parameters should be taken into consideration as variable; the others may be treated as constants through their mean values.

2. STRUCTURAL MODEL FOR ANALYSIS

The model of a member under the weight of structure and topping and live load - q, is presented in Fig. 1 [5].

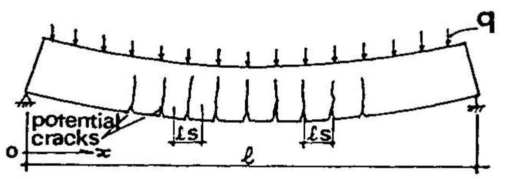


Fig. 1: Model of member with potential cracks.

The probability of crack appearance in a member with a particular mean concrete strength - f_{cm} under the considered loads, according to the accepted model, is:

$$P_q = 1 - \prod_{o}^{\ell/\ell_s} (1 - P_x) \qquad (1)$$

where :

ℓ - member span,

ℓ_s - spacing of potential crack sites, predetermined by experiments [5], ℓ_s = 40 mm.

P_x -probability of crack appearance in the x-section, for a Gaussian distribution:

$$P_x = \sum_{j=-\infty}^{j_o} [\exp(-j^2/2)] S_c / \sqrt{2\pi} \tag{2}$$

S_c - summation step .

The limit of summation j_o in (2) is evaluated by :

$$j_o = [(M_{xp}/W_1 + M_{xq}/W + f_{p1} - f_p)/f_{cm} - 1]C_{v1} \tag{3}$$

where :

C_{v1} - variation coefficient of concrete strength in a member over its length,

f_{p1} - tensile stress due to topping concrete shrinkage,

f_p - residual prestress in most stressed fibre,

W_1 - section modulus of considered prestressed member,

W - section modulus of member with topping,

M_{xp} - moment in the x-section due to dead loads,

$$M_{xp} = M_{xg} + M_{xt} \tag{4}$$

M_{xg} - moment due to dead weight (g) of prestressed part of structure,

$$M_{xg} = g/2\ (\ell x - x^2)\ ,$$

M_{xt} - moment due to dead weight of topping (g_t), $M_{xt} = g_t/2\ (\ell x = x^2)$,

M_{xq} - moment due to live hoad q, $M_{xq} = q/2\ (\ell x - x^2)$;

$$M_{xp} = (g + g_t) \cdot (\ell x - x^2)/2 \tag{5}$$

q - sum of given live loads,

f_{cm} - mean strength of concrete in considered member.

3. ESTIMATION OF CRACK APPEARANCE IN A MEMBER IN THE STRUCTURE POPULATION

The probability of crack appearance in a member (Fig. 1) with a given grade of concrete under a particular load is evaluated by :

$$\sum_c (P_q \cdot P_1) \tag{6}$$

where

c - number of summation steps for given concrete grade (f_{cmm} - mean strength for this grade),

$$f_{cm} = f_{cmm}(1 + i_o \cdot C_{vo}) \tag{7}$$

P_1 - probability of occurrence of the particular mean concrete strength in the considered member, namely

$$P_1 = S_1 \; [\exp(i_o^2 / 2)]\sqrt{2\pi} \tag{8}$$

where

S_1 - summation step, i_o - independent parameter, C_{vo} - variation coefficient of the mean concrete strength in members. The overall variation coefficient of tensile strength :

$$C_v = \sqrt{C_{v1}^2 + C_{vo}^2} \; ,$$

by [1] : $C_v = 18.3\%$.

The overall cracking probability of the considered structure in the general case, with all variables taken into account, may be computed by :

$$P = \Sigma(\Sigma(\Sigma(\Sigma(\Sigma(\Sigma(P_q \cdot P_1) \cdot P_2) \cdot P_3) \cdot P_4) \cdot P_5) \cdot P_6) \tag{9}$$

where the subscripts 1-6 of the partial probabilities refer specifically to : (1) mean concrete strength, (2) ressidual prestress, (3) member weight, (4) topping weight, (5) topping shrinkage, (6) live loads, respectively.

4. NUMERICAL ANALYSIS OF CRACKING IN PRESTRESSED SLABS

The different floor slabs (Fig. 2) are analysed for the characteristic live load (q_k) determined by the traditional approach as the live load causing crack appearance in the middle section of the considered member :

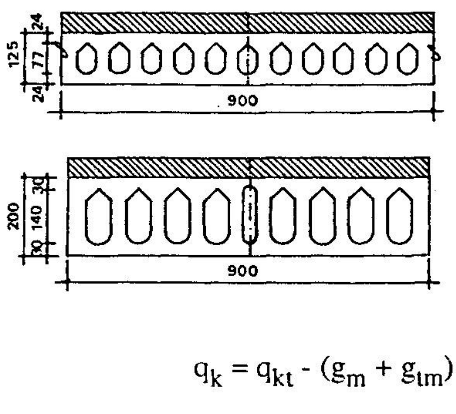


Fig. 2: Sections of analysed floor slabs.

$$q_k = q_{kt} - (g_m + g_{tm}) \; W / W_1 \tag{10}$$

where

q_{kt} - total load causing crack appearance in the member :

$$q_{kt} = 8\,(f_{pk} + f_{cmk} - f_{p1}) \times W / \ell^2 \qquad (11)$$

f_m and g_{tm} - dead weights of prestressed element and of topping, respectively.

f_{pk} - design residual prestress in the most stressed fibre.

f_{cmk} - characteristic concrete flexural-tensile strength.

f_{p1} - stress in above-mentioned fibre due to concrete shrinkage of the topping.

W - section modulus of composite slab.

W_1 - section modulus of prestressed element.

ℓ - slab span.

The essential characteristics of the analysed slabs are the following: concrete strengths - C50 - of prestress elements, C - 30 - of toppings; spans - 6 m and 9 m respectively for section depth 125 mm and 200 mm. The live loads were varied within the limits of the variation coefficient C_q, from .05 to .3 . The residual prestress was varied in limits of C_p, from .02 to .15 . The coefficients of variation of concrete stress were included in the analysis by [1]: $C_{vo} = C_{v1} = .13$, the weight variations by $C_v = .1$ and topping shrinkage by $C_{v,sh} = .15$. The essential computation results are shown in Figs. 3(a) and (b).

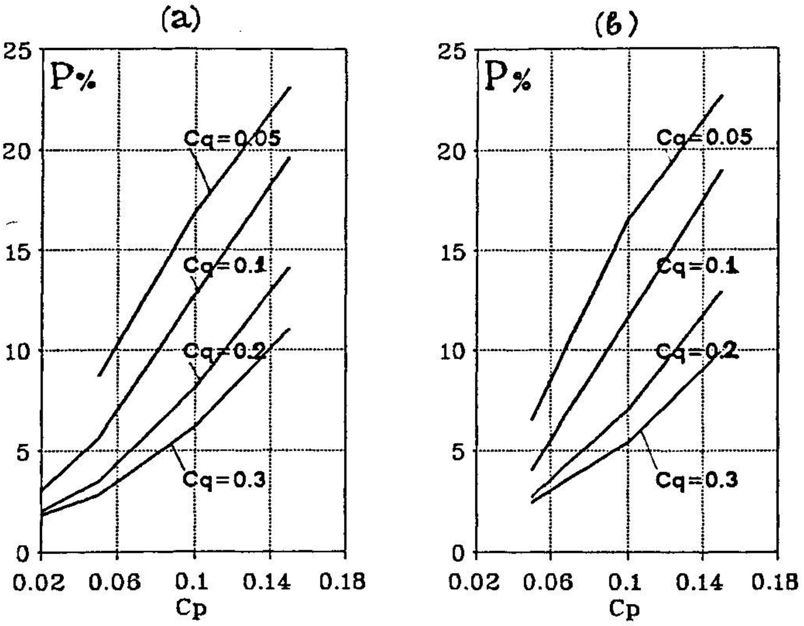


Fig. 3: Crack appearance probabilities in 6m-span slabs (a) and in 9m span slabs (b) vs. variation coefficient of presstress (C_p) for different variations of live load.

It is seen that under the traditional approach we can have in practice a wide range of reliability in terms of cracking.

The variability of member- and topping weights as well as of topping shrinkage affects the crack appearance probabilities only slightly. Variation of the concrete strength, residual prestress and live load may significantly affct the service reliability in terms of cracking. In principle, if all parameter variations obey the codes, the structural reliability in terms of cracking should be very low. The situation, it seems, is improved through such favorable factors as : (1) superior real concrete strength with lower variability and (2) lower real live load.

5. CONCLUSIONS

Traditional estimation of concrete structure servieability in terms of cracking depending on multiple parameters yields significantly differing reliabilities for structures with different variable parameters. The probability of crack appearance in analysed common prestressed multihollow slabs with given codified parameters may range from a few percent to 20% and even more. At the same time, in actuality, some favourable factors, such as higher concrete strength, lower strength variability and lower loads can increase the structural reliability.

The main problem is to arrive at equally reliable structures. The best solution is suitable probabilistic analysis based on probabilistic criteria and statistical initial data. Such initial parameters as concrete stength, prestress and live loads should be taken into account as variables. Other parameters can be taken as constants, by their mean values.

In any event, the traditional cracking estimate should be corrected by behaviour factors, taking into account the load-and prestress variations, as well as the variable characteristics of the concrete strength and real possible deviations of the essential structure parameters.

All the foregoing calls for supplementation of the codes by suitable probabilistic restrictions and by statistical data on loads and strength of materials.

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Probabilistic Design Concept

Concepts probabilistes de calcul

Probabilistische Konzepte der Berechnung

Milan HOLICKÝ
Klokner Institute
Czech Techn. Univ.
Prague, Czech Rep.

Lars ÖSTLUND
Professor
Lund Inst. of Technology
Lund, Sweden

Milan Holický, born 1943, got his civil engineering and doctor degree at the Czech Technical University in Prague, his doctor degree also at the University of Waterloo, Canada. Milan Holický, is now active in research and standardization of structural reliability.

Lars Östlund, born 1921, got his civil engineering degree at the Royal Institute of Technology, Stockholm. He was professor in Structural Engineering at Lund Institute of Technology from 1965 to 1986. At present he is convener of CIB W81 working group Action on Structures.

SUMMARY
Two kinds of uncertainties are to be generally distinguished when analyzing structural serviceability: randomness of basic variables and vagueness in definition of limit states. The second kind of uncertainty may be handled by methods of fuzzy set theory. Derived unserviceability measures enable one to formulate probabilistic design concepts including optimization.

RESUME
L'analyse de l'aptitude au service des systèmes porteurs des bâtiments implique de distinguer en général deux sortes d'incertitudes: le caractère aléatoire des variables de base et de la définition imprécise des états limites. Il est possible d'appliquer la théorie des ensembles flous au dernier type d'incertitude. Les critères en découlant, et définissant une aptitude au service défectueuse, permettent de formuler des concepts de dimensionnement probabilistes par l'application d'une méthode d'optimisation.

ZUSAMMENFASSUNG
Bei der Beurteilung der Gebrauchstauglichkeit von Gebäuden sind im allgemeinen zweierlei Unsicherheiten zu unterscheiden: die Zufälligkeit der Basisvariablen und die Unschärfe in der Definition der Grenzzustände. Letztere kann mit Methoden der Fuzzy-Set-Theorie behandelt werden. Daraus abgeleitete Kriterien mangelnder Gebrauchstauglichkeit erlauben die Formulierung wahrscheinlichkeitstheoretischer Bemessungskonzepte unter Einsatz von Optimierungsverfahren.

1. INTRODUCTION

Serviceability of building structures is their ability to perform adequately in normal use [1,2]. It is well recognized that due to several trends in modern design and construction, serviceability of building structures is becoming more and more important economic as well as technical issue [3,4,5,6]. Moreover, current procedure for dealing with serviceability are from various reasons insufficient and need to be improved.

One of the most important tasks is an identification of relevant functional requirements and their specification in terms of suitable set of serviceability parameters u_i. General guidance is offered in another contributions [7,8] at that colloquium. It appears that more requirements are often to be considered simultaneously, and both structural response to actions and deviations due to production procedures are to be considered. Nevertheless, in most cases only one serviceability parameter u is considered at a time (for example deflection at midspan, slope, amplitude, acceleration). In some cases, however, two or more parameters are to be investigated simultaneously (for example deflection and amplitude, amplitude and acceleration).

The most frequently applied serviceability criteria limit the actual values of serviceability parameter u_i, denoted $z_i(t)$, t being time, by time independent limiting values l_i [7]; in case of single parameter u the following inequality is traditionally used

$$z(t) \leq l \tag{1}$$

This condition may be generalized for more complex quantities and/or a set of parameters u_i, actual values $z_i(t)$ and limiting values l_i. However, the fundamental question to be clarified first concerns rational and rigorous definition of the quantities entering any serviceability condition including the fundamental one, described by Equation (1).

2. UNCERTAINTIES

It is well recognized [6,9,10] that structural response $z(t)$ in Equation (1) depends on a number of basic variables of random nature such as actions, material properties and geometrical quantities. Consequently $z(t)$ is a random function of the time t, which may have considerable variability. Generally the structural response may be described by probability density function $\varphi_z(u,t)$, the mean function $\mu_z(t)$ and standard deviation function $\sigma_z(t)$, which become constants when structural response is described by time independent random variable z. In some cases probability distribution of structural response is not symmetrical and in that case skewness (likely to be positive) could be used [10].

The limiting value l on the right hand side of Equation (1) generally follows from functional requirements, which are often expressed in qualitative (verbal) way only and, consequently, are very subjective. Thus, the limiting values are also affected by considerable uncertainties, partly of a different nature than those involved in structural response $z(t)$. Evidently, in serviceability limit states in is rarely possible to distinguish unambiguously between acceptable and unacceptable state. This imprecision or vagueness in definition of limit states, appears to be the most significant source of great differences in evaluation and practical assessment of structural serviceability.

Evidently, there are two kinds of uncertainties to be considered when analyzing structural serviceability: randomness of basic variables or resulting variables and vagueness or imprecision in definition of serviceability limit states. While more familiar randomness of variables can be handled mathematically through the well established theory of probability, less familiar imprecision and vagueness in definition of serviceability limit states may be handled by methods of newly developing theory of fuzzy sets [11,12,13]. The following theoretical model for limiting values l_i of serviceability parameters is based on both concepts: randomness and fuzziness.

3. SERVICEABILITY LIMITS

Consider the fundamental case of a single serviceability parameter u (for example deflection at midspanpoint or amplitude). It is assumed that with increasing parameter u, the ability of a structure to comply with specified functional requirements decreases and level of serviceability damage increases. In some cases a single distinct value l_0 could be identify, which separates unambiguously acceptable and unacceptable state. This, rather special case, may be described by stepwise membership function $\mu_S(u)$, shown in Figure 1. As a function of the serviceability parameter u it indicates membership of a structure in a set S of serviceable structures

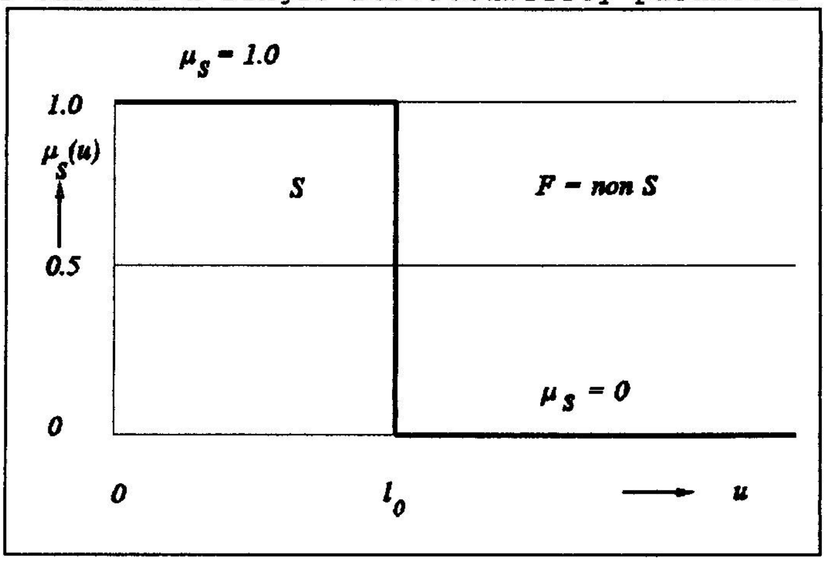

Figure 1 Membership function $\mu_S(u)$.

$$\mu_S(u) = 1\,, \quad \text{if} \quad u < l_0\,,$$
$$\mu_S(u) = 0\,, \quad \text{if} \quad u \ge l_0\,, \tag{2}$$

Generally, however , the membership function $\mu_S(u)$ may be more complicated [14]. A conceivable and more realistic form of the function $\mu_S(u)$ could be

$$\mu_S(u) = 1\,, \qquad \text{if } u < l_1\,,$$
$$\mu_S(u) = \frac{(l_2 - u)^n}{(l_2 - l_1)^n}\,, \qquad \text{if } l_1 \le u < l_2\,, \tag{3}$$
$$\mu_S(u) = 0\,, \qquad \text{if } l_2 \le u\,,$$

which is shown in Figure 2. Transition region, where the structure is gradually becoming unserviceable is specified by the lower limit l_1 and the upper limit l_2. Both these limits together with the exponent n characterize vagueness or fuzziness of the limit state and should be derived from its nature. Fuzzy set of unserviceable (damaged or failed) structures F is the complement of the set of serviceable structures S, thus F = *non* S. The membership function of the set F is given [11,13] as

$$\mu_F(u) = 1 - \mu_S(u). \tag{4}$$

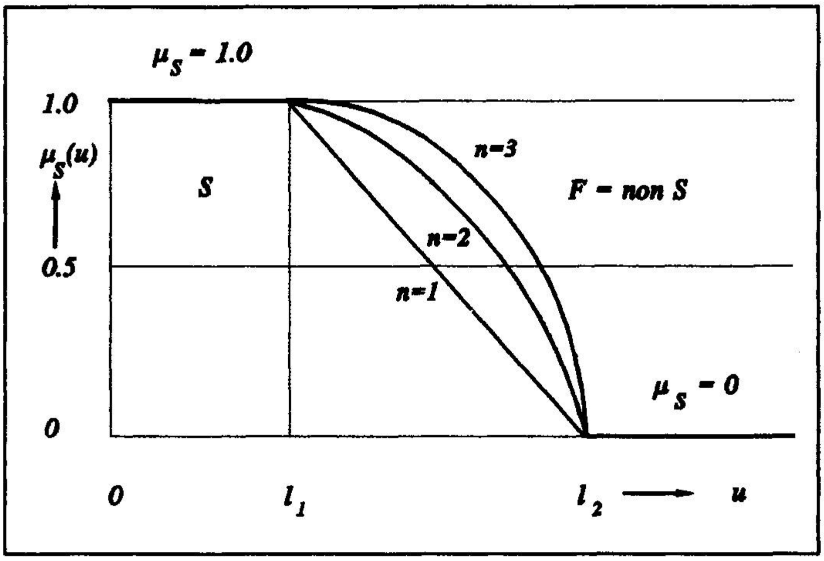

Figure 2 Membership function $\mu_S(u)$.

Furthermore, for a given serviceability level μ_S (function symbols without arguments are used to denote a variable or numerical value), serviceability parameter u (including both limits l_1 and l_2), may have considerable scatter. Similarly for a given parameter u, serviceability level μ_S may be a random variable. This randomness (not fuzziness) of membership function is caused by natural variability of human perceivability to various defects or due to random deviation in properties of installed machinery or secondary structures [6]. Therefore, the membership functions $\mu_S(u)$ and $\mu_F(u)$ are generally random functions of the serviceability parameter u. Variability of the membership function $\mu_F(u)$ for n = 1, which is the case used in the following analyses, is indicated in Figure 3. It is assumed, that above defined membership functions $\mu_F(u)$ represents the mean function and, furthermore, that for any given damage level μ_F, the probability density function of the serviceability parameter u may be described by normal distribution $\varphi_F(u'/\mu_F)$ having the mean equal to u'', for which $\mu_F(u'') = \mu_F$, and approximately constant (at least in a relevant interval of the parameter u) standard deviation σ_F.

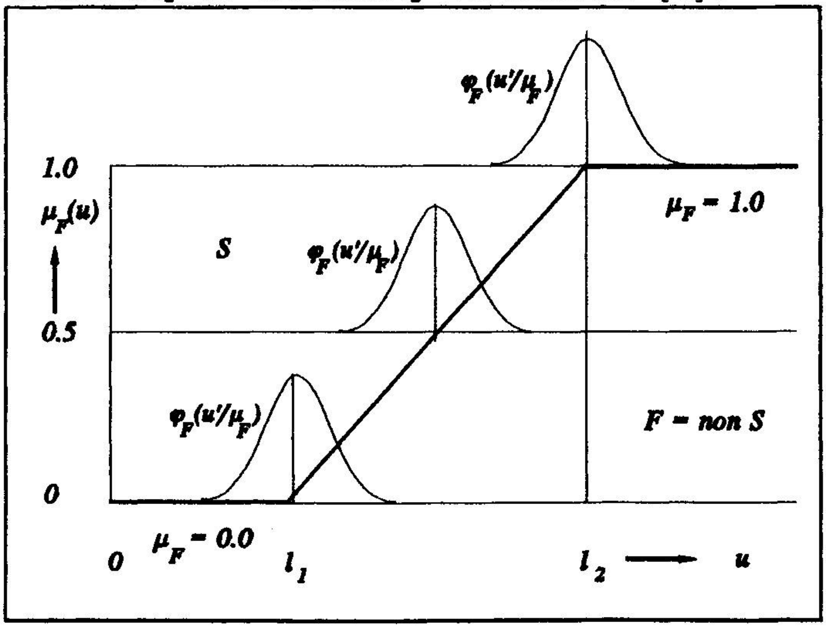

Figure 3 Membership function $\mu_F(u)$.

The above theoretical model of serviceability limits is consequently characterised by fuzziness characteristics l_l, l_t and the exponent n, and by the randomness characteristic represented by the standard deviation σ_f. Four extreme combinations of both concepts may be obviously recognised:

(a) deterministic case, when $l_l = l_t = l_0$, and $\sigma_f = 0$,
(b) pure fuzziness, when $l_l \neq l_t$ and $\sigma_f = 0$,
(c) pure randomness, when $l_l = l_t = l_0$ and $\sigma_f \neq 0$,
(d) fuzzy-random case, when $l_l \neq l_t$ and $\sigma_f \neq 0$.

From the most general combination of both concepts (d), which is treated bellow, the other combinations may be obtained by appropriate choice of the model characteristics. For example the case of pure randomness (c), which is considered in [6], is obtained for $l_l = l_t = l_0$.

4. UNSERVICEABILITY MEASURES

Expected unserviceability at a given damage level μ_F is the cumulative function $\Phi_F(u/\mu_F)$ of the serviceability parameter u,

$$\phi_F(u/\mu_F) = \int_{-\infty}^{u} \varphi_F(u'/\mu_F)\, du'. \tag{5}$$

The total expected unserviceability (damage) corresponding to the serviceability parameter u is defined as weighted expected unserviceability with respect to all possible damage levels μ_F

$$\phi_F(u) = \frac{1}{N} \int_0^1 \mu_F\, \phi_F(u/\mu_F)\, d\mu_F\,, \tag{6}$$

where $N = 1/(n+1)$ is the normalizing factor to limit the total unserviceability into the interval $\langle 0,1\rangle$. The limiting value l can be now defined as the parameter u for which the total expected unserviceability it is equal to a required value Φ_F, thus

$$\phi_F(l) = \frac{1}{N}\int_0^1 \mu_F\, \phi_F(l/\mu_F)\, d\mu_F = \phi_F\,. \tag{7}$$

Taking into account random character of structural response $z(t)$, the probability of failure of a structure at a given damage level μ_F and time t is provided by the integral

$$p_f(\mu_F, t) = \int_{-\infty}^{\infty} \varphi_z(u/t)\, \phi_F(u/\mu_F)\, du, \tag{8}$$

where $\varphi_z(u/t)$ denotes the probability density function of structural response $z(t)$. The total instantaneous unserviceability with respect to all possible damage levels μ_F at the time t is

$$p_f(t) = \frac{1}{N}\int_0^1 \mu_F\, p_f(\mu_F, t)\, d\mu_F\,, \tag{9}$$

The total probability of failure p_f within the whole intended life time T is then

$$p_f = \frac{1}{T}\int_0^T p_f(t)\,dt\,. \tag{10}$$

The above unserviceability measures, given by Equations (7), (8), (9) and (10), can be used to formulate various types of design criteria.

Moreover, if the actual malfunction cost of any structure is proportional to the damage level μ_F, then the expected malfunction cost C_f can be expressed [16] as

$$C_f = \frac{1}{T}\int_0^T C_F(t)\,p_f(t)\,dt \approx p_f\,C_F\,. \tag{11}$$

where the malfunction cost $C_F(t)$ due to the full unserviceability (when $\mu_F = 1$) is approximated by a time independent value C_F. If the total cost C could be expressed as the sum of the initial cost C_0 and expected cost C_f given by Equation (11), thus

$$C = C_0 + p_f\,C_F. \tag{12}$$

then optimization procedure may be applied [15,16]. Necessary conditions for the minimum total cost follows from partial derivatives with respect to optimization variables.

5 EXAMPLE

The following example is based on experimental data [17], concerning serviceability limit state of visual disturbance. Excessive sags of *49* reinforced concrete floors and beams were recorded when annoying deformations were perceived. Observed disturbing sags z/L, where L denotes span of horizontal components [4], are within a broad range from *0.003* to *0.018*. Using this data the mean membership function $\mu_F(u)$, may be approximated by the tri-linear function ($n=1$), indicated in Figure 3. Further, the following fuz-

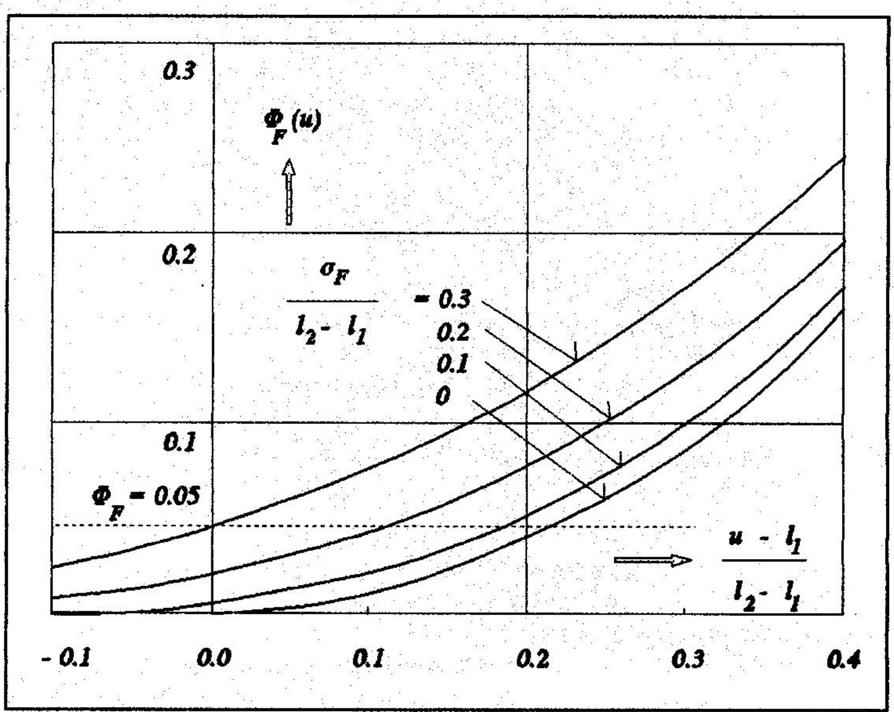

Figure 4 The function $\Phi_F(u)$ for $n = 1$.

ziness and randomness characteristics were derived from these data

$$\frac{l_1}{L} = 0.003, \quad \frac{l_2}{L} = 0.014, \quad \sigma_f = 0.05\,(l_1 - l_2). \tag{13}$$

The standard deviation σ_f was assessed from scatter of the data about the mean function as one twentieth of the transition length.

The total expected unserviceability $\Phi_f(u)$ for $n = 1$ is shown in Figure 4. It follows from Equation (7) and Figure 1, that for $\Phi_f = 0.05$ the limiting deflection is $l \approx l_l + 0.2\,(l_2 - l_l) = L/192$, if $\Phi_f = 0.01$, then $l \approx l_l + 0.05\,(l_2 - l_l) = L/282$. It should be however noted, that used experimental data do not include all the relevant information, and some additional assumptions were required to define the above model. More data, supplemented by information on level of observed damage, are urgently needed.

Let the cross section height h, be a single optimization variable. The sag z may be expressed as $z = K\,h^{-3}$, where K denotes a constant. If the initial cost $C_0(h)$ is proportional to h, then the first derivative of Equation (12) yield the condition [5]

$$\frac{C_0(h)}{C_F} = 3\mu_z \frac{\partial p(\mu_z, \sigma_z)}{\partial \mu_z} + 4\,\sigma_z \frac{\partial p(\mu_z, \sigma_z)}{\partial \sigma_z}. \tag{14}$$

The mean sag μ_z, determined for selected ratios C_f/C_0 and coefficients of variation σ_z/μ_z using Equation (14) and characteristics described by Equations (13) are given in Table 1.

Table 1. The optimum mean sag μ_z/L

Ratio σ_z/μ_z	Ratio C_f/C_0				
	1	5	10	100	1000
0.00	1/159	1/251	1/282	1/391	1/498
0.05	1/181	1/316	1/376	1/571	1/781
0.10	1/220	1/431	1/532	1/855	1/1205
0.20	1/313	1/680	1/847	1/1351	1/2041

If the coefficient of variation $\sigma_z/\mu_z = 0.10$, then the optimum mean μ_z equals $L/220$ for $C_f/C_0 = 1$, $L/532$ for $C_f/C_0 = 10$. It appears, that commonly applied limiting values of the range from $L/360$ to $L/200$ correspond to relatively low cost of full malfunction C_f ($C_f/C0$ from 1 to 5) and high fuzzy probability of failure p_f (from 0.01 to 0.05). Consequently, commonly accepted serviceability constrains may be frequently uneconomical.

5. CONCLUSIONS

(1) Two kinds of uncertainties are to be distinguished when analyzing structural serviceability: randomness and vagueness.

(2) Imprecision and vagueness in definition of structural serviceability may be handled by methods of fuzzy set theory.

(3) Proposed unserviceability measures enable to formulate probabilistic concepts for design of structural serviceability

including optimization.

(4) Optimization of serviceability limit state due to visual disturbance indicates, that commonly used limiting values for sag of horizontal components may be uneconomical.

(5) Further research is recommended to concentrate on
- experimental data enabling more accurate theoretical models for vagueness in definition of limit states,
- fuzzy concept for multidimensional serviceability problems.

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Australian Performance Standard for Domestic Metal Framing

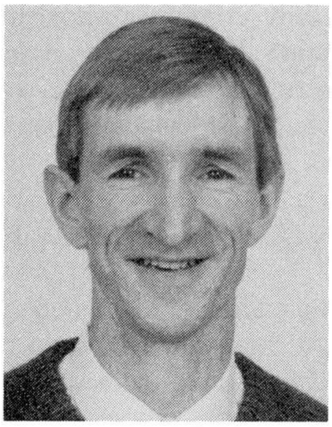
Normes de qualité australiennes pour des bâtiments à ossature métallique

Australische Güteanforderungen an Stahlskelettwohnbauten

Lam PHAM
Senior Princ. Res. Scientist
CSIRO
Melbourne, Australia

Lam Pham Ph.D., FIE Aust. has been active in research and drafting of Australian Standards in building construction. His main areas of interest are loadings (load combinations, snow and earthquake loads), applied structural reliability, steel and timber design and domestic construction.

Graeme STARK
Senior Research Officer
BHP Steel
Wollongong, Australia



Graeme Stark M.E, MIE Aust. works on building systems research and development within the cold-formed steel industry in Australia. One of his main areas of interest has been domestic steel framing systems, including the drafting of design manuals and standards for the design of steel framed houses.

SUMMARY

This paper presents the background of the serviceability requirements of the Australian Standard on Domestic Metal Framing. This is the first Australian structural performance standard to include serviceability as part of the mandatory requirements. The difficulties involved in drafting the serviceability requirements are discussed. Details of the requirements are given. These included serviceability performance under static loads, dynamic performance of floors and verification procedures.

RESUME

Cet article présente les données de base des exigences d'aptitude au service relatives aux normes de qualité australiennes pour des bâtiments à ossature métallique. Pour la première fois en Australie, l'aptitude au service a été prise en compte en tant qu'exigence obligatoire dans une norme qualitative sur les structures. Les auteurs rappellent les difficultés survenues au cours de l'étude préliminaire. Ils donnent les détails des dispositions correspondantes, qui tiennent compte de l'aptitude au service sous charge statique, du comportement dynamique des planchers et de méthodes de vérification.

ZUSAMMENFASSUNG

Der Beitrag schildert den Hintergrund für die Gebrauchstauglichkeitsanforderungen der australischen Norm für Stahlskelettwohnbauten. Damit wird erstmals in einer australischen Tragwerksnorm die Gebrauchstauglichkeit als bindende Anforderung aufgenommen. Die beim Entwurf aufgetretenen Schwierigkeiten und Einzelheiten der Bestimmungen werden geschildert. Diese beinhalten die Gebrauchstauglichkeit unter statischer Belastung, das dynamische Verhalten von Geschossdecken und Nachweisverfahren.

1. INTRODUCTION

The introduction of performance standards is part of Australian Standards policy of developing multi-part standards, with the first part being the performance requirements and subsequent parts being deemed-to-comply solutions. The Performance Standard for Domestic Metal Framing [1] is one of the first of this new generation of standards.

This paper presents the background of the structural serviceability requirements of this Standard. It is the first Australian Structural Standard to include serviceability as part of the mandatory requirements. Aspects of the Australian domestic metal framing industry are briefly outlined to explain the needs for a performance standard in this area and the reasons to make serviceability requirements mandatory. General aspects of performance standards and serviceability requirements are discussed. These include the needs of various users of the standard such as the industry, the owners and the building control authorities; and the difficulties in developing a rational serviceability specification. Details of the proposed serviceability specification are then described. These include static serviceability loads as well as serviceability limits for roof, wall and floor systems and the dynamic performance of floors. The problems of verification are also discussed.

2. AUSTRALIAN DOMESTIC METAL FRAMING INDUSTRIES

Most houses built in Australia are of framed construction, with timber framing dominating. Metal-framed construction, although it has been in existence for more than 30 years in Australia, constitutes only a small fraction of the houses built. It dominates the kit-home market and is popular for construction in remote areas where building materials are difficult to obtain. Recently, it has gained more popularity with the project builders.

Although the term 'metal framing' is used so that aluminium is not excluded from the Standard, at present all metal framed houses being built in Australia are made of cold-formed light-gauged steel. The steel components are the roof trusses, the wall frames and the floor joists. The components may be used separately with other traditional construction materials such as timber or brick or together in an all steel-framed house. A steel-framed house may have a metal or tiled roof, brick veneer or hardboard-clad external walls, and plasterboard on internal walls and ceilings. In a finished house it is difficult to identify the type of framing.

Almost all Australian metal framing is based on proprietary systems. Most systems have adopted different section shapes to suit their particular designs, since roll-formed steel framing component can be of almost any shape and dimension. A large number of innovative developments are currently taking place in Australia as the market share for metal framing increases. To assist in the development of cold-formed steel framing for domestic construction, a document titled 'Structural Performance Requirements for Domestic Steel Framing' [2] was produced by the authors for the industry as a forerunner of the Standard. The performance standard has been drafted at the request of the industry to create a fair competitive environment not only between different metal-framing systems but also between different construction materials eventually. Mandatory structural serviceability requirements are also the wish of the industry to ensure some degree of uniformity in performance between different competing steel framing systems.

3. STRUCTURAL SERVICEABILITY AND PERFORMANCE STANDARD

'A performance standard describes all the features that are required of a product but does not prescribe what to do to attain those features. It offers means of verification that the product will behave as intended. It allows the selection and the comparison of products for a particular purpose from the widest possible range, consistent with the need of the user' [3].

The drafting of the serviceability performance standard for the Australian domestic metal-framing industry has to take into account the needs of various interested parties other than the industry, such as the owners/occupiers and the building control authorities. Serviceability problems, in the perception of the occupiers, represent quality defects, although quality assurance and serviceability are two separate issues and should be dealt with separately. The users need a performance specification that is independent of the material of construction so that they can be assured of satisfactory performance regardless of their choice of material. The designers need clearly stated serviceability conditions that can be assessed preferably by computation or simple deemed-to-comply requirements. The building control authority needs performance criteria that are easily verifiable.

While general aspects of performance are easily identified, e.g. roofs should not sag and walls should not crack, etc., they are not easily quantified. Acceptable frequencies of exceeding serviceability limit states may vary through several orders of magnitude depending on the type of limit state considered, the variability of the human response to a serviceability condition and the cost associated with providing a certain level of serviceability. Another difficulty is to define structural serviceability conditions and to relate them to actual building performance. The behaviour of domestic construction is complex because of the system effects which are difficult to account for in design. Serviceability criteria should be developed based on cost-effectiveness concepts. A reliability model could be developed to include all sources of variability and uncertainty, particularly the variability in the people's responses to serviceability phenomena together with relative costs associated with providing certain levels of serviceability. From the model, the most cost-effective solution and the corresponding serviceability criteria can be derived. While the theoretical framework for such a model is available [4], its application requires considerable input data and is not yet available.

The Standard committees' immediate aim is to develop a serviceability performance specification which is:

- simple to use;
- based on well defined structural parameters that are measurable and computable;
- easily understood by the designers; and ideally
- independent of the construction materials

... The drafting committee has therefore adopted the following strategies:

- basing the requirements on the levels currently accepted for domestic construction in Australia as exemplified by existing construction; and
- limiting the requirements to those identifiable with specific aspects of performance and verifiable preferably with in-situ measurements.

4. FEATURES OF THE PROPOSED SERVICEABILITY SPECIFICATION

4.1 General

The proposed draft standard requirements are limited to structural serviceability performance. Both the load or the load combination and the serviceability limit appropriate for a specific serviceability condition are given together. The criteria given are intended to give satisfactory performance for most domestic construction because they are based on the performance of currently accepted construction. For specific situations, they may be varied if found inappropriate. An appendix to the Standard gives typical examples of these situations. The reasons for the performance requirements are also given in the appendix, while the standard proper refers only to specific requirements.

4.2 Static Performance

4.2.1 Performance under dead loads

Out-of-flatness deflections under dead loads are restricted to prevent objectionable sagging and possible damage to architectural finishes. The general limit is span/300 with different absolute limits for different components. This is applicable to roof battens, roof rafters, (with an absolute limit of 20 mm), ceiling joists (12 mm) and lintels (9 mm). Roof truss top chords are expected to have the same performance as roof rafters and bottom chords as ceiling joists. Better performance is expected of ceiling battens with a limit of span/600. Top plate deflection under dead load can only occur if there is no alignment of the roof trusses or rafters with the studs. For this situation, a limit of span/240 or 6 mm has been imposed for top plates in single or upper storeys and a limit of span/300 for lower storeys. For floors, the permanent gravity load consists of dead load and the sustained component of live load (set at 40% of the design live load). For this combination a static deflection limit of span/250 has been set for both floor joists and bearers.

All the above limits are based on the levels currently accepted for domestic construction using currently accepted design procedures [5].

4.2.2 Performance under live loads

Defection under live load has been used in this serviceability specification to control the stiffness of members to ensure adequate performance. Performance under two types of live loads has been specified. For a concentrated live load of 1.1 kN representing the weight of a person, a deflection limit of span/180 has been set for roof batten to prevent tile cracking due to a person walking on a roof. For roof trusses, a limit of span/270 has been used for deflection between truss panel points with an absolute value of 15 mm for maintenance purposes. For members in the lower storey of a two-storey construction, a deflection limit of span/200 has also been placed for the design live load of 1.5 kPa. This limit is applicable to top plates and lintels.

4.2.3 Performance under wind loads

A serviceability wind load has been specified by the Australian Wind Loading Standard corresponding to a wind speed which has a 5% chance of exceedance in any one year. For studs supporting flexible wall cladding, a deflection limit under wind of span/240 with a maximum of 12 mm has been set. For stiffer wall cladding such as ceramic tiles, a tighter limit of span/360 with a maximum of 8 mm has been imposed. No limit is placed on studs in a brick veneer construction. As the brick veneer skin is much stiffer than the wall frame, the serviceability wind pressure is not likely to be transferred to the stud wall. Traditionally other deflection limits have been placed on various other components but they have been deliberately omitted from this performance specification because no rational basis for them has been found.

4.3 Dynamic Performance

The specification for dynamic performance has been limited to floor systems. The standard has not yet placed a performance requirement for accidental impact loads on walls although the need for such a requirement has been discussed.

Dynamic performance of floors is a difficult problem. Parameters that affect dynamic performance and methods of measuring and specifying these parameters are still subject to considerable discussion, although progress has been made in the understanding of the problem [6]. Australian steel joist floors, for various practical reasons, are built over a fairly limited range of parameters. Joist spacings are either 450 or 600 mm, timber decking is either 19 or 22 mm thick. Over these ranges, satisfactory performance has been obtained for C-joist design using span/750 limit on deflection under a specified uniformly distributed live load of 1.5 kPa. For rectangular hollow sections, successful design has been obtained with span/500 as the dynamic performance criterion. These are however deemed-to-comply criteria. Effort has been made to relate the performance of these floors to dynamic parameters, the account of which is given in another paper at this colloquium [7].

At the time of writing this paper, the committee has not made up its mind over various available options for specifying dynamic performance:

(a) limiting acceleration induced by a foot fall;
(b) limiting peak velocity due to an impact load;
(c) limiting deflection due to a unit concentrated load; and
(d) maintaining the traditional method of limiting deflection under uniformly distributed load.

Option (c) is theoretically the weakest but it has been shown to work in practice. For any of the other three options to be used, a calibration exercise has to be carried out to relate the criteria to the currently acceptable floors.

4.4 Verification

The Standard provides two methods for the verification of a particular design for its serviceability performance: by computation or by testing.

For verification by computation, the load redistribution caused by the system effects may be taken into account. The Standard, however, offers little guidance on load redistribution except for the grid effects on concentrated and partial area loads.

For verification by testing, the Standard only provides guidance for prototype testing which is useful in developing new framing systems. Because of the complex system behaviour, testing is not only feasible but also often the most economical way to verify the serviceability performance of a steel frame subassembly or component.

5. CONCLUSION

The background and the main features of the structural serviceability requirements of the draft Australian Performance Standard for Domestic Metal Framing have been presented. Although the draft still has many shortcomings, it will fulfil the basic need of the metal framing industry of ensuring some degree of uniformity in the structural performance of steel framed houses.

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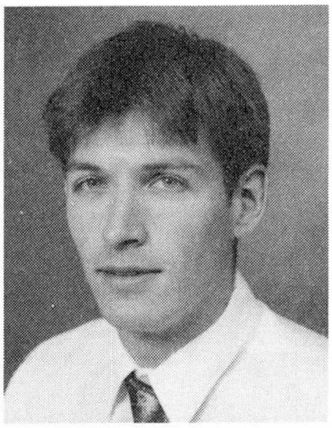
Role of Load Testing and Structural Monitoring in Appraisal

Rôle des essais de charge et étude structurelle lors des évaluations

Rolle der Belastungsprüfung und Tragwerksüberwachung bei der Bewertung

Richard MOSS
Higher Scientific Officer
Building Research Establishment
Watford, England



Richard Moss, born 1963, obtained his first degree at Sussex University and his Ph.D. from Imperial College, London University. During his period of 8 years at BRE, his research has concentrated on load testing although he has recently initiated research on structural monitoring and has carried out general appraisal work.

SUMMARY

This paper considers the role of load testing and structural monitoring in appraisal. Problems associated with each of these techniques are identified and possible solutions explored. Particular problems identified in relation to load testing are the interpretation of the results and practical difficulties of carrying out the tests. The author concentrates in particular on enhancement in stiffness produced by non-structural screeds, and results from a large programme of work testing beam and block concrete floors are discussed. In overcoming practical difficulties dynamic testing is put forward as a possible alternative to static load testing in certain circumstances. Problems associated with monitoring schemes are also considered.

RESUME

Le présent exposé examine le rôle des essais de charge et de l'étude structurelle lors des évaluations. Les problèmes normalement associés à chacune de ces techniques sont ici identifiés, et les éventuelles solutions explorées. Parmi les problèmes particuliers associés aux essais de charge, il faut mentionner l'interprétation des résultats et les difficultés de nature pratique, à savoir la réalisation proprement dite desdits essais. L'auteur met l'accent sur la rigidité produite par les revêtements, et examine les résultats d'un vaste programme d'essais de planchers en béton formés de parpaings ou de poutres. En vue de résoudre les difficultés pratiques, il est fait mention des essais dynamiques en tant qu'alternative possible aux essais de charge statique dans certaines situations. Par ailleurs, sont également abordés les problèmes associés aux projets de contrôle.

ZUSAMMENFASSUNG

Der Beitrag behandelt die Rolle der Belastungsprüfung und Überwachung bei der Tragwerksbewertung. Es werden mit diesen Verfahren verbundene Probleme herausgestellt und mögliche Lösungen gesucht, Ausdeutung der Ergebnisse und praktische Schwierigkeiten bei der Ausführung der Prüfung. Der Verfasser konzentriert sich besonders auf die Steifigkeitserhöhung, die sich aus nichttragenden Anstrichen ergibt, und diskutiert die Ergebnisse eines grossen Arbeitsprogramms, in dessen Rahmen Balken- und Massivbetondecken geprüft wurden. Zur Lösung der praktischen Schwierigkeiten wird unter gewissen Umständen eine dynamische Prüfung als Alternative zur statischen Belastungsprüfung vorgeschlagen. Mit der Überwachung verbundene Probleme werden ebenfalls behandelt.

1. INTRODUCTION

1.1 The average age and number of existing structures is increasing with time and structures, particularly concrete structures, are subject to deterioration mechanisms which can eventually result in impaired structural performance. In the future there will therefore be an increasing burden of maintenance and repair and an increase in demand for structural re-evaluation, central to which is the appraisal and assessment of structures.

1.2 The sources of information available to an engineer when carrying out an appraisal on an existing structure are:

a. Existing documentation on the original design and construction and any subsequent modifications.

b. The maintenance history of the structure.

c. Surveys of the structure providing information on:

1. As-built dimensions, reinforcement details etc.
2. Present loadings (from re-assessment of current dead and imposed loads).
3. The physical condition and properties of the construction materials.
4. Any visible defects.

2. THE RÔLE OF LOAD TESTING AND STRUCTURAL MONITORING IN APPRAISAL

2.1 Other techniques in addition to the above can be used, in particular load testing and structural monitoring.

2.2 Load testing involves the application of test loads to a structure and measurement and interpretation of the response of the structure to those loads. Full-scale load tests are normally very expensive and time-consuming to carry out. However, there are some structures which are not amenable to calculation and in such circumstances the only way to make an assessment is to carry out load tests.

2.3 Where there is a change of use of a structure or for some other reason there is doubt as to the structural adequacy of the construction, a subsequent approach to carrying our conventional structural assessment is to install a monitoring scheme. Structural monitoring is a developing field and there is a need to develop an understanding of what can be achieved by monitoring. Research is also required to develop the methodology and hardware systems for in-service monitoring of building structures.

2.4 Application of appropriate assessment and monitoring techniques can provide justification for extended building life with potentially very substantial cost savings.

3. PROBLEMS ASSOCIATED WITH LOAD TESTING

3.1 Interpretation of results

3.1.1 The main problem associated with load testing is interpretation of the results from tests since correct interpretation relies on a proper understanding of the behaviour of structures.

3.1.2 In his research the Author has addressed specific issues in relation to load testing of floor and roof structures [1]. The need for further research on load testing was identified in the light of the results of investigations into the use of high alumina cement concrete (HACC) construction [2]. HACC was used extensively to manufacture floor beams used in beam and pot type floor and roofing systems.

3.1.3 The particular problems addressed by the Author are the assessment of the effects of load distribution and, secondly, the assessment of the influence of movements resulting from temperature variations upon the load induced deformations.

3.1.4 To solve these problems the Author has developed methods using linear theory and heat conduction analysis leading to assessment of load distribution, thermal deflections and load corrections.

3.1.5 The Author has completed a large programme of work testing beam and block floors looking at the influence of different types of floor screed on the structural behaviour. This work has demonstrated the very considerable increase in stiffness due to non-structural screeds and this is described further in Section 4.

3.2 Practical and logistical difficulties

3.2.1 The other main problem associated with carrying out full-scale static load tests is the time, inconvenience and expense associated with them. In contrast to static testing, dynamic testing, although requiring specialist equipment and personnel, is much quicker and easier to carry out, and hence the possible role of dynamic testing in load testing procedures has been explored.

3.2.2 From the dynamic tests carried out attempt has been made to predict behaviour under static loads from measured dynamic characteristics. The results available so far suggest that a reasonable estimate of the extent of lateral load distribution can be made, but that the magnitude of the deflection is not predicted very well as illustrated in Figure 1.

3.2.3 Dynamic testing may have a potential role to play in selecting test areas and also assessing boundary conditions. However by its very nature dynamic testing can only provide an insight into behaviour at load levels generating a linear response.

4. ENHANCEMENTS IN STIFFNESS DUE TO NON-STRUCTURAL SCREEDS

4.1 Details of tests on beam and block floors

4.1.1 A large programme of testing of beam and block floors has been carried out, principally up to and slightly beyond service loads. This is so that at each stage of construction the load-deflection curves obtained were repeatable, and the additional stiffening effect produced by that stage of construction assessed. The stages at which the floor was tested were:-

1. Individual beams
2. Beams and blocks (ungrouted)
3. Beams and blocks (grouted)
4. Beams and blocks (grouted) plus floating screed finish
5. Beams and blocks (grouted) plus unbonded screed finish
6. Beams and blocks (grouted) plus bonded screed finish

4.1.2 The boundary conditions of the floor were varied, and lateral restraint to transverse movement of the floor was found to have some influence on the stiffening effect produced and its reliability.

4.1.3 Tests on 11 nominally identical precast concrete beams showed there to be considerable variability in stiffness between them (± 10%).

4.2 Enhancements in measured beam stiffness

4.2.1 The term beam stiffness is here used to refer to the ratio of the load carried by a beam (as measured by its end reactions) to its central deflection.

4.2.2 The stiffness of an individual beam tested in the grouted floor was increased by about 20% when the floor was restrained in the transverse direction, and about 10% when unrestrained.

4.2.3 For the floor with different screed types with transverse restraint, the average stiffness increases were 75%, 37% and 360% for the floating, unbonded and bonded screed respectively. For the floor without transverse restraint the corresponding values were 85%, 36% and 270%. These stiffness increases correspond to reductions in deflections of about 40%, 25% and 75% respectively. Not surprisingly the stiffness increase produced by the bonded screed is considerably greater than that for the unbonded and floating screeds and this is illustrated in Figure 2.

4.2.4 The differences in measured stiffness increases of the beams were reflected in calculations which showed that only a very small width of floating screed (14mm) needs to be acting compositely with a beam to produce the stiffness increase observed, whereas the width of bonded screed needed is 225mm. The beam spacing was 500mm.

4.2.5 The Author is currently developing a model for assessing the stiffness increase produced by different types of non-structural screed, taking account of slippage at the interface between the screed and the other components.

4.2.6 The stiffening effects described above need to be considered when interpreting the results of load tests.

4.3 Reliability of stiffness increases determined

4.3.1 The stiffness increase produced by a floating screed was found to be dependent on how well the screed was bedded down onto the rest of the floor structure, and this must raise doubts as to what extent the stiffening effect of a floating screed could be relied upon in practice.

4.3.2 When testing the floor with a bonded screed cracking occurred in the screed above one of the beams, reducing its stiffness. This most likely resulted from lack of ability of the screed to resist tensile stresses induced as a result of differential movement between the beams and shrinkage of the screed after being laid. The reliability of a bonded screed could perhaps be increased by incorporating a nominal mesh within the screed to help resist tensile stresses.

4.3.3 Such cracking would be more likely to occur for a beam and block floor than for some other types of floor construction (e.g. hollow plank), because of greater tendency for outwards horizontal movement. However tests to 1.25 x design service moment which have recently been conducted on a hollow plank floor assembly have revealed a similar effect.

4.3.4 The extent to which the stiffening effects can be relied upon will depend not just upon on how reliable the interaction mechanism is, but also on whether the physical presence of the screed can be guaranteed. In such cases there is clearly a need for redefinition of what can be classed as 'structural' and what is 'non-structural'. Enhancements to stiffness provided by different screed types could eventually be taken account of in the design process, although much greater attention would then need to be focused on the specification of the 'non-structural' materials.

5. PROBLEMS ASSOCIATED WITH STRUCTURAL MONITORING

5.1 The problems associated with monitoring schemes can be divided into four broad categories. These are:

a. Defining the objectives of the scheme

b. Selection of positions to monitor

c. Instrumentation and system performance

d. The limitations of monitoring systems in warning of sudden distress.

5.2 There are many reasons for installing a monitoring scheme but they can be broadly categorised as:

a. Where modifications to existing structures are being carried out (strengthening, demolition etc)

b. Where long-term movements are required to be monitored (eg due to ASR, temperature changes, ground movements etc)

c. Where structures are subject to ongoing corrosion damage or other forms of deterioration

d. For research purposes (ie to provide a direct feedback loop to the design process in terms of providing a better understanding of structural behaviour and the actions to which structures are subjected)

e. Where accurate assessment of fatigue life is required (eg for bridges and offshore structures)

f. Where a novel system of construction is being employed (eg use of alternative prestressing materials and spaceframes).

6. PROGRAMME OF RESEARCH ON STRUCTURAL MONITORING AT BRE

6.1 The programme of research currently in hand at BRE aims to tackle the problems identified above. Work is being carried out under contract which will help to develop a methodology for deploying monitoring instrumentation based on concepts of robustness and vulnerability.

6.2 Reviews of case histories of structural monitoring and instrumentation have been completed [3], [4] and small-scale trials of instrumentation are under way.

6.3 The review of instrumentation which has been carried out in parallel with the review of case histories has identified the parameters which it is desirable to measure, the most appropriate instrumentation to use for measuring these parameters, and the most appropriate data logging system in which to integrate the instrumentation.

6.4 The instrumentation which has been considered has in general been restricted to that which is capable of being incorporated within data logging systems so that measurements can be taken automatically and remotely. Such a system is essential where large numbers of instruments required to be read within a relatively short time span, or alarms are to be activated.

6.5 In the review future developments such as the use of expert systems and active structural control are also considered.

6.6 There is a potential rôle for expert systems in aiding the interpretation of data obtained from a monitoring scheme, and such information could ultimately be used to control the response of a structure, for example under earthquake or other extreme loading conditions.

6.7 Expert systems work on the premise that there is a considerable data bank of existing knowledge and expertise. In many cases this data bank will not be available for structural monitoring applications and in these circumstances the expert system would need to be developed over a considerable period of time based on experience with the particular structure concerned.

6.8 In his review the author concludes that it is not practicable to formulate practical instrumentation systems for different applications. Rather the approach he suggests is to have a 'tool-kit' of available instrumentation from which to choose the best instrumentation for any particular application.

6.9 Recommendations are given on the most promising instrumentation devices to form part of this 'tool-kit' and the most suitable data logging system in which to integrate them. These recommendations have formed the basis for the small-scale trials currently in hand.

6.10 For long-term monitoring (ie over many years) the reliability and stability of the instrumentation is of crucial importance and one of the main objectives of the small-scale trials is thus to test out the long-term performance of different types of instrumentation.

7. CONCLUSIONS

1. Attempts at predicting static behaviour from measured dynamic characteristics have met with some success. A reasonable estimate of the extent of lateral load distribution could be made, but the magnitude of the deflections was not predicted very well.
2. Dynamic testing may have a rôle to play in selecting test areas and assessing boundary conditions. However by its very nature it can only provide an insight into behaviour at load levels generating a linear response.
3. For beam and block floors non-structural screeds, particularly bonded screeds, laid over the top surface will have a very considerable stiffening effect.
4. The reliability of the stiffness enhancement will vary between different types of screed, and the enhancement produced by floating and bonded screeds may be less reliable than for unbonded screeds.
5. Enhancements in stiffness produced by non-structural screeds need to be taken account of when

assessing the performance of existing floor construction. Such enhancements might eventually be taken account of in the design process, although much greater attention would then need to be focused on the specification of the 'non-structural' materials.

8. ACKNOWLEDGEMENTS

The author would like to acknowledge the assistance of Mr D Brooke in testing of the beam and block floors with unbonded and bonded screeds, and in the preparation of the figures.

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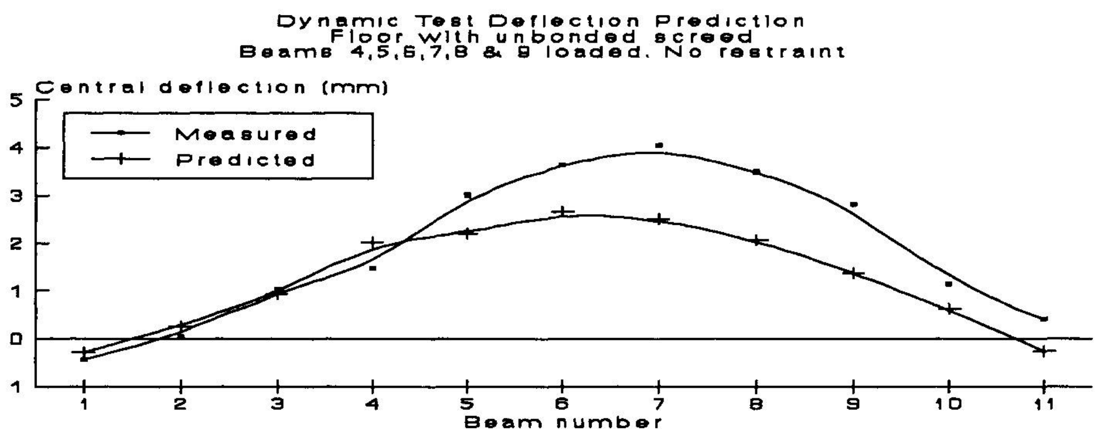


Figure 1: Comparison of measured deflections and deflections predicted from dynamic analysis for beam and block floor with unbonded screed.

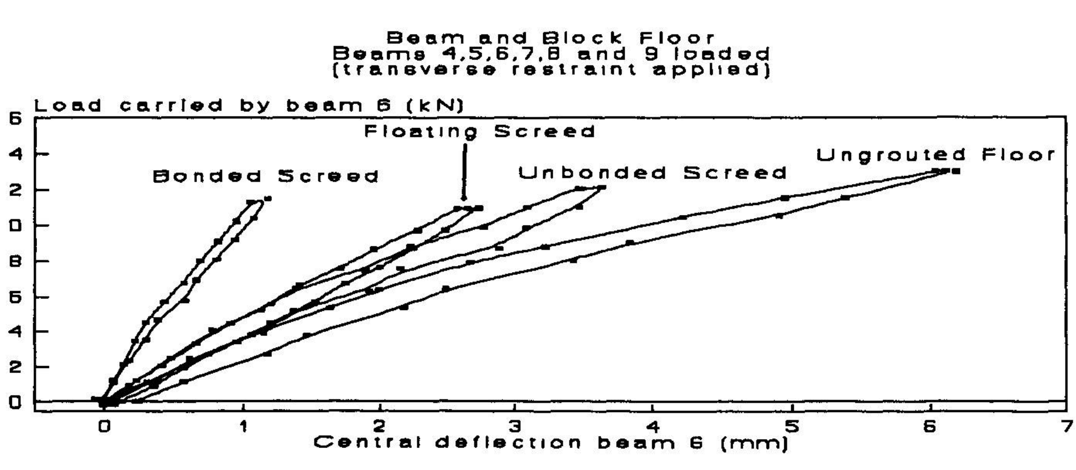


Figure 2: Influence of screed type on beam stiffness. (Transverse restraint applied)

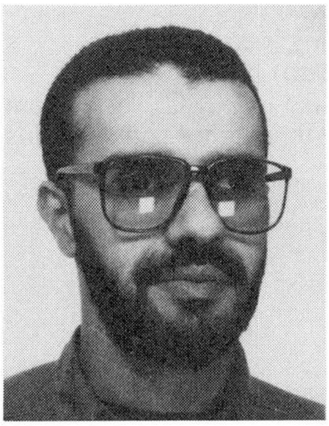
Stuctural Serviceability of Buildings

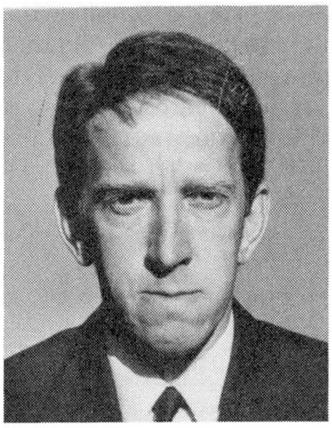
Aptitude au service des bâtiments

Gebrauchstauglichkeit von Gebäuden

Messaoud SAIDANI
Research Fellow
University of Nottingham
Nottingham, UK

David A. NETHERCOT
Prof. of Civil Engineering
University of Nottingham
Nottingham, UK





SUMMARY

The paper reports on the main findings as they relate to the provision of deflection limits for serviceability design. A review has been conducted that has shown that numerous serviceability design criteria exist but that these are spread diversely through codes, papers, journal articles, technical reports, standards, or are simply the customary practice of individual engineers.

RESUME

Cette publication présente les principaux résultats obtenus concernant les valeurs acceptables de limites de déformation pour le dimensionnement en service. Une recherche bibliographique montre qui'il y a un bon nombre de critères de dimensionnement en service qui existent déja mais ceux-ci sont diversement éparpillés dans des codes, publications, articles de journaux, rapports techniques, normes, ou sont simplement le fruit du travail d'ingénieurs isolés.

ZUSAMMENFASSUNG

Es werden die Hauptergebnisse einer Übersicht über vorhandene Vorschriften zu Durchbiegungsbeschränkungen für die Gebrauchstauglichkeit vorgestellt. Wie sich zeigte, existieren zahlreiche Kriterien, sind aber in diversen Normen oder anderen Vorschriften, technischen Berichten und Aufsätzen verstreut, sofern sie nicht bloss dem Erfahrungswissen des einzelnen Ingenieurs entspringen.

1. INTRODUCTION

Increasing adoption of limit states based approaches to the design of steel structures has tended to concentrate researcher's attentions on the need to reliably predict load levels corresponding to the attainment of the structure's ultimate static strength. Thus design is based on scientific studies that ensure a suitable margin against plastic collapse, buckling, fatigue failure etc. Although codes and specifications also call for checks at serviceability, these are usually couched in rather simple terms and little real guidance on exactly how such checks be conducted or exactly what they are intended to achieve is provided. There is thus at least the suspicion of a considerable imbalance between the qualities of design for the ultimate condition and design for the serviceability condition.

It was in recognition of this that a three-part programme of research, focusing on static deflections of steel framed buildings, funded by the European Coal and Steel Community (ECSC), was started in late 1990. It comprised of:
- Investigation of the in-service performance of steel buildings (TNO-Bouw)
- Review of existing code requirements and their basis (University of Nottingham)
- Numerical studies and consideration of design models (University of Trento).

A report [1] giving the findings of each aspect of the work has been presented to ECSC. The content of this paper is based on the code review section and is complemented by three other papers at this conference which deal with the other topics.

2. SERVICEABILITY IN CONSTRUCTION

2.1 Problems associated with excessive static deformation

In modern construction a number of problems associated with limit states related to excessive static deformation (deflection, settlements, rotation, curvature, drift etc.) can be identified. Some of the most common are:
- local damage to non-structural elements (eg. ceilings, partitions, walls, doors and windows, etc.) due to deflections caused by load, temperature variation, shrinkage or creep, and moisture changes
- deterioration of the structure by fatigue
- discomfort due to vibrations (produced by use of machines, traffic, etc.)

noticeable deflections causing distress to occupants.

An acceptable structural design must ensure that such problems are properly identified and their occurrence minimised. The use of suitable materials, properly connected components (through efficient bolting and welding), allowing for thermal expansions by providing sufficient separation between deflecting primary structural elements and non-structural components, are all factors that should be addressed.

2.2 Economic aspects

Limiting deflections to an appropriate level is also an important issue as far as economy is concerned. In a recent seminar on "Serviceability limit states for steel buildings" held in Zürich [2], Golembiewski presented a report on this matter. He showed that the limit of h/150 for the lateral deflection of hall structures due to wind, and adopted by the Swiss Steel Construction Standard SIA 161 [3], is a severe demand. A value of h/100 was suggested as being sufficient. This was based on the results of many years of experimental research undertaken in the old GDR, which showed that with this limit, damage was not to be expected. As stated by Golembiewski this difference is in fact significant, since sharpening h/100 to h/150 requires up to 15% more steel in the case of heavy roof claddings and up to 35% in the case of light-weight roof claddings.

(a) Deflection of beams due to unfactored imposed load	
Cantilevers	Length/180
Beams carrying plaster or other brittle finish	Span/360
All other beams	Span/200
(b) Horizontal deflection of columns other than portal frames due to unfactored imposed and wind loads	
Tops of columns in single-storey buildings	height/300
In each storey of a multi-storey building	height of story under consideration/300
(c) Deflection of crane gantry girders	
Vertical deflection due to static wheel load	span/600
Horizontal deflection (calculated on the top flange properties alone) due to crane surge	Span/500

Table 1 Deflection limits for certain structural members in accordance with the British BS 5950 : Part 1 [4]

Type of beam	Deflection to be considered	Deflection limit d for span L(1)	Deflection limit d for cantilever L(2)
beam supporting masonry partitions	deflection which occurs after the addition or attachment of partitions	$\delta/L \leq 1/500$ where provision is made to minimise the effect of movement, otherwise, $\delta/L \leq 1/1000$	$\delta/L \leq 1/250$ where provision is made to minimise the effect of movement, otherwise, $\delta/L \leq 1/500$
all beams	total deflection	$\delta/L \leq 1/250$	$\delta/L \leq 1/125$

Table 2a Suggested vertical deflection limits for beams (AS 4100 [5])

Notes:

(1) Suggested deflection limits in this table may not safeguard against ponding.
(2) For cantilevers, the values of δ/L given in this table apply, provided that the effect of the rotation at the support is included in the calculation of δ.

Building clad in steel or aluminium sheeting gantry cranes and without internal partitions against external walls	$\frac{1}{150}h$
building with masonry walls supported by steelwork	$\frac{1}{240}h$

Table 2b Suggested horizontal deflection limits (AS 4100 [5])

3. SERVICEABILITY LIMIT STATES IN CURRENT CODES

Design rules for serviceability may be found in the Standards of many countries. They should, as indicated above, ensure a balance between acceptable performance in service and economical considerations. A full review is available [1]; the following provide some idea of present coverage:

• The British code BS5950:Part 1:1990 [4] makes a provision for serviceability limit states design. Two types of limit states are considered: deflection and durability. For the latter, the code suggests that the following factors should be considered at the design stage:
- the environment
- the degree of exposure
- the shape of the members and the structural detailing
- the protective measure if any
- whether maintenance is possible.

Table 1 lists vertical as well as horizontal deflection limits for beams, columns, and gantry girders. In addition to the fact that this section of the code is advisory, private discussions with engineers in the UK showed that serviceability is rarely considered in design.

• The Australian code AS4100-1990 [5] gives recommendations for vertical deflection limits for beams and horizontal deflection limits for buildings -Table 2. These recommendations, like those of BS5950, are advisory and do not cover a number of serviceability aspects.
A comparison between BS5950 and AS4100 shows that for beams (in general) the deflection limit is L/200 in BS5950 and L/250 in AS4100, which represents a difference of 22% (with BS5950 being more conservative)- see Table 4.

• In the draft European EC3: 1991 code, section 4 on "Serviceability limit states" [6], a description of some serviceability requirements for steelwork is given. These cover:
- deformations and deflections which affect the appearance or effective use of the structure.
- vibration, oscillation or sway which causes discomfort to the occupants of a building or damage to its contents.
- damage to finishes or non-structural elements due to deformations, deflections, vibration, oscillation or sway.

The code, however, does not cover some important aspects of serviceability, e.g. cladding effect on lateral deflections, differential settlements etc. In addition it does not specify the load combination for a particular deflection limit. As specified in section 4.2.1 of the code, the deflection limits are empirical and should not be interpreted as performance criteria. It is worth noting that the limits specified in BS5950 agree well with those in EC3, with the latter appearing to be more specific (see Table 4).

• In 1988 the Building Research Association of New Zealand (BRANZ) published a technical report containing research work undertaken by Cooney and King [7] intended to assist structural engineers establish suitable deflection criteria, in order to ensure serviceability of buildings. The report reviews the following items:
- reasons for limiting deflections
- effect on structural elements
- effect on sensory acceptability
- effect on use
- prevention of damage to non-structural elements.

In addition, the report analysed the sensitivity of deflection components with regard to:
- section modulus
- changes in section
- component end restraint and rotation effects
- loading assumptions (distribution and intensity)
- shear distortions etc.

Reason for limiting deflections	Deflection limitations	Load combination	Examples and Comments
water accumulation (ponding) on roofs etc.	$\delta < L/250$ for beams parallel to line of roof slope	D (allow for creep) plus rainwater or snow melt	
beams that support surfaces which should drain water	$\delta < L/250$ $\delta < L/350$ $\delta < L/600$	D + L or D + S D + L or D + S D or D + S	–reinforced concrete or steel beams supporting slabs –trafficable deck supported by timber beams –non-trafficable deck supported by timber beams (always check that water flows as designed) L = live load; D = dead load; S = snow load
differential settlement	$\delta < L/300$ $\delta < L/150$		–beams supporting masonry walls –beams supporting walls other than masonry

Table 3a Examples of limiting deflection values for horizontal components in the BRANZ Manual, New Zealand [7]

Reason for limiting deflections	Deflection limitations	Load combination	Examples and Comments
sway of columns due to wind	$\delta < h/500$ and < 4mm per storey	D + W	applies especially to multi-storey buildings D = dead load; W = wind load
frame deflection due to wind and earthquake	horizontal deflection at eaves $\delta <$ L/200 x frame spacing; and < 40 mm in end bay	W	W = wind load
differential settlement	$\delta < h/300$ $\delta < h/150$		–masonry –other material

Table 3b Examples of limiting deflection values for vertical components in the BRANZ Manual, New Zealand [7]

Tables 3a and 3b give some deflection limits for typical components (see also Table 4 for comparison with other codes).

• A translation of a Dutch document on serviceability requirements [8] has been provided by the CISTI (Canada Institute for Scientific and Technical Information) [9]. In summary, the report recommends the following for the effects of static deformations and their allowable values:

- water accumulation (on roofs): it can be prevented by judiciously determining the point of water discharge.
- subjective aspect: becomes more significant if the deformations become visible.
- use aspect: this is to ensure permanent serviceability of the floor structure. Requirements depend on each individual situation and there is no general rule.
- construction aspect: floor and roof static deformations in structures may give rise to cracking or other damage in members which are supported by these structures (a typical example is the cracking in partitions). As a recommendation for beams or floors supported on two or more ends, the following conditions were suggested:

$\delta_{add}/L \leq 500$ to 600 L = span parallel to the partition wall

and also $\delta_{add} < 10$ to $20\,mm$

where, δ_{add} = additional deflection occurring after installation of the wall

Before closing this section it is worth mentioning that the CIB (Conseil International du Batiment) has launched a review exercise designated W85 dealing with structural serviceability [10]. It is mainly concerned with phenomena such as deformations, vibrations and damage to non-structural components. The findings of the research should be available by 1993.

4. COMMENTS

It is clear from the extract from the review [1] given in the previous section that the present treatment of just one aspect of serviceability design – the provision of deflection limits given in steel building codes – is not presented in a consistent fashion world-wide. This contrasts with attempts to base strength design on more of a common treatment e.g. use of the multiple column curve concept. It is believed that the deflection issue is, however, actually less clearly provided for, than cursory examination of the evidence would suggest.

The reason for this is the potential for significant differences between "true" and "design" treatments of each of these quantities:

- loading
- model used for calculations
- limiting criteria

This review has looked only at the third of these but the real issue is:

> What deflection limit is appropriate for use with the set of design loads used for the serviceability condition and the method employed for calculating such deflections in order that the actual structure loaded by its in service loading does not suffer unsatisfactory performance?

Clearly there is a link between loading – model – limit. Thus the information presented herein should be accepted within the context of the wider study [1]. Only by examining true behaviour and design-type check calculations for a range of building types can a suitable design package, that will ensure acceptable in service behaviour of the real structure, emerge.

5. CONCLUSION

The investigation carried out on the serviceability requirements has shown the importance of the issue. A review, undertaken for a limited number of codes, showed the complexity of the issue when considering the limiting criteria to be used in the design for serviceability. It is clear from the extracts from the review that present treatment of just one aspect of serviceability design –the

Code	Deflection limits		
	Beams in general	Tops of columns in buildings	
		Single storey	Multi storey
BS5950	L/200	h/300	h/300
AS4100	L/250	h/240	-
EC3	L/200	h/300	h/300
BRANZ	L/250	-	-

Table 4 Deflection limit examples in different codes

provision of deflection limits given in steel building codes– is not treated in a consistent fashion worldwide. This contrasts with attempts to base strength design on more of a common treatment basis. It is believed that the deflection issue is, however, actually less clearly provided for than even this cursory examination of the evidence would suggest.

6. ACKNOWLEDGEMENTS

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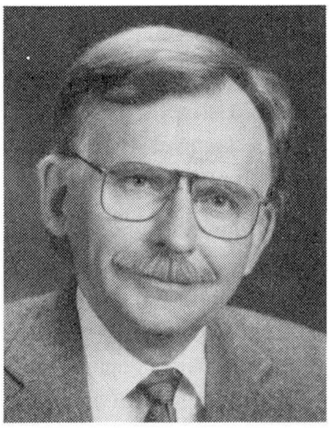
Floor Vibrations: A New Design Approach

Nouvelle façon d'étudier la vibration des planchers

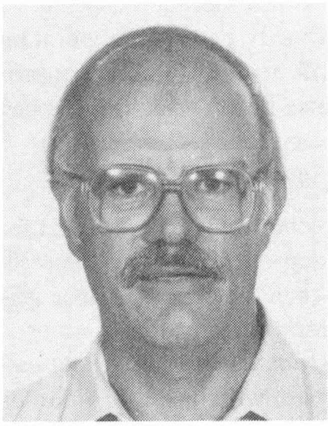
Eine neue Lösung für Deckenschwingungen

Thomas M. MURRAY
Prof. of Struct. Steel Design
Virginia Polytechnic Institute
Blacksburg, VA, USA

David E. ALLEN
Senior Researcher
National Research Council Canada
Ottawa, ON, Canada



Thomas M. Murray, born 1940, obtained a Ph.D. from the University of Kansas in 1970. He has held faculty positions at the University of Oklahoma and the US Air Force Academy. He is currently the Montague-Betts Professor of Structural Steel Design, Virginia Polytechnic Institute and State University.



David Allen, born 1934, obtained a Ph.D. from University of Illinois in 1966 and has ben a researcher with the National Research Council Canada ever since. He has authored about sixty papers in diverse subjects related to structural engineering, including safety criteria human error, fire resistance and floor vibration.

SUMMARY

A new criterion for walking vibrations with broader application is discussed. The criterion is based on the dynamic response of steel structures to walking forces. The criterion can be applied to offices, shopping malls and footbridges.

RESUME

Un nouveau critère, ayant un plus vaste domaine d'application, pour l'étude de la vibration des planchers sous l'effet des piétons, est présenté. Le critère est basé sur l'étude de la réaction dynamique des structures métalliques quand elles sont soumises à des forces causées par la marche des piétons. Le critère peut être utilisé pour l'étude de la vibration des passerelles et des planchers dans les bureaux et dans les centres commerciaux.

ZUSAMMENFASSUNG

Ein neues Kriterium für Schrittschwingungen, mit breiter Anwendung, wird vorgelegt. Es basiert auf der dynamischen Reaktion von Stahlkonstruktionen auf Schrittkräfte. Das Kriterium kann auf Geschäftsgebäude, Einkaufszentren und Fussgängerbrücken angewendet werden.

1. INTRODUCTION

Existing North American floor vibration design criteria are usually based on a reference impact, such as a heel-drop, and were calibrated using floors constructed 20-25 years ago. Annoying floors of this vintage generally had natural frequencies between 5 and 8 hz because of the then existing design rules and common construction practice. With the advent of LRFD and the more common use of lightweight concrete, floor systems have become lighter, resulting in higher natural frequencies for the same structural steel layout. Beam and girder spans, however, have increased resulting in frequencies lower than 5 hz. Most existing design criteria do not properly evaluate systems with frequencies below 5 hz and above 8 hz.

A new criterion for walking vibrations with broader applications has recently been proposed [1]. The criterion is based on the dynamic response of steel beam and joist supported floor systems to walking forces. The criterion can be used to evaluate structural systems supporting offices, shopping malls and footbridges. This paper provides an overview of the proposed criterion.

The reaction of people who feel vibration depends very strongly on what they are doing. People in offices or residences do not like distinctly perceptible vibration (about 0.5 percent g), whereas people taking part in a non-stationary activity will accept vibrations approximately 10 times greater (about 5 percent g or more). People dining beside a dance floor, lifting weights beside an aerobics gym, or standing in a shopping mall, will accept something in between (about 2 percent g). Sensitivity within each occupancy, however, varies with duration of vibration and remoteness of source.

Most floor vibration problems are due to repeated forces caused by machinery or by human activities such as dancing, aerobics or walking. In some cases the repeated force is sinusoidal, or nearly so, although walking is a little more complicated than the others because it is not a stationary force. A repeated stationary force can be represented by a Fourier combination of sinusoidal forces with forcing frequencies equal to a multiple (harmonic) of the basic frequency of force repetition (step frequency for human activities). As a general rule, the sinusoidal forces decrease with increasing harmonic, more so if a large number of people are involved. If any of the harmonic forces correspond with the natural frequency of a susceptible vibration mode, then resonance will occur. Walking can excite resonance for more than one harmonic, but experience shows that a consideration of resonance with a lower mode is sufficient for design.

2. OVERVIEW OF PROPOSED CRITERION

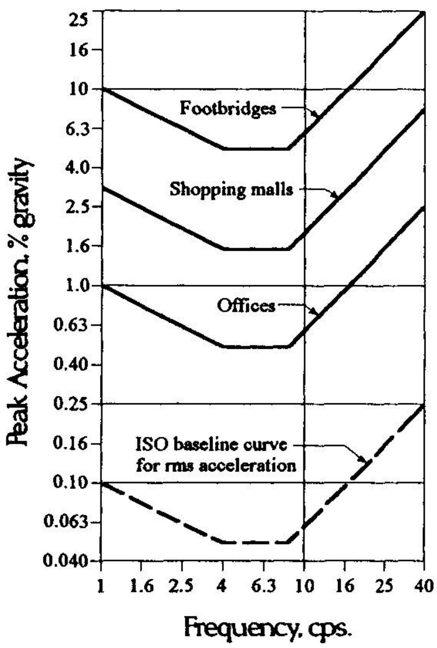


Figure 1 Proposed Acceleration Limits for Walking Vibrations

The proposed criterion was developed using the following:

- Acceleration limits as recommended by the International Standards Association (2) adjusted for intended use. The ISO suggests limits in terms of rms acceleration as a multiple of the baseline line curve shown in Figure 1. The multipliers in the proposed criterion are 10 for offices, 30 for shopping malls, and 100 for footbridges.

- A time dependent loading function repesented by the Fourier series

$$F = P(1 + \Sigma\alpha_i \mathrm{Cos}2\pi ift) \tag{1}$$

where P is the person's weight, taken as 0.7 kN for design, f the step frequency, i the harmonic multiple, and α_i the dynamic load factor for the harmonic. Proposed values for α_i are given in Table 1.

- A response function of the form:

$$\frac{a}{g} = \frac{R\alpha_i P}{\beta W} \bullet \mathrm{Cos}2\pi ift \tag{2}$$

Harmonic i	Frequency Range $i \bullet f$	Dynamic Load Factor α_i
1	1.6 to 2.2	0.5
2	3.2 to 4.4	0.2
3	4.8 to 6.6	0.1
4	6.4 to 8.8	0.05

Table 1 Dynamic Load Factors

where W is the total weight supported by the beam, β is the damping ratio, g is the acceleration due to gravity, and R is a reduction factor. The reduction factor R takes into account the fact that full steady-state resonance is not achieved for walking and that the walking person and the person annoyed are not simultaneously at the location of maximum modal displacement. It is proposed that R be taken as 0.7 for footbridges and 0.5 for floor structures having two-way modal configurations.

The proposed criterion is obtained from Eqns. (1) and (2) expressed as a minimum value of damping ratio times equivalent mass weight

$$\beta W \geq \frac{R\alpha_i P}{a_o / g} \tag{3}$$

As shown in Reference [1], Inequality [3] can be approximated as

$$\beta W \geq K \exp(-0.35 f_o) \tag{4}$$

where f_o is the fundamental natural frequency (Hz) and K is a constant which depends on the acceleration limit for the occupancy: 58 kN for offices, 20 kN for shopping malls and 8 kN for footbridges. In terms of minimum fundamental frequency, Inequality [4] is

$$f_o \geq 2.86 \ln\left[\frac{K}{\beta W}\right] \tag{5}$$

Inequality (5) is then the proposed criterion for floor vibrations due to walking. That is, an acceptable floor is one with a natural frequency greater than the right side of Inequality (5).

The following section provides guidance for estimating the required floor properties for application of the proposed criterion.

3. ESTIMATION OF REQUIRED PROPERTIES

Recommended values for the damping ratio, β, are 0.03 for offices, 0.02 for shopping malls, and 0.01 for footbridges. If full height partitions are connected at the top and at the bottom to the structure, the damping ratio for offices can be increased to 0.05. A value of 0.02 should be used if few non-structural components (ceiling, ducts, partitions, etc.) are supported by an office floor. These values are modal damping ratios and are approximately one-half of previously recommended damping values which were based on vibration decay resulting from a heel-drop impact [3,4].

The fundamental natural frequency, f_o, and equivalent mass weight, W, for a critical mode are estimated by first considering the 'joist panel' and 'girder panel' modes separately and then combining them. If the joist span is less than one-half the girder span, both the joist panel mode and the combined mode should be checked separately. (For the purposes of this paper, a "joist" is a structural member supported by a girder; a girder is a structural member supported by a column or wall.)

The first natural frequency for the joist and girder panel modes can be estimated using

$$f_o = 0.18 \sqrt{g / \Delta} \tag{6}$$

where Δ is the maximum deflection of the joist or girder due to the weight supported by the member. Composite action is normally assumed, provided there is sufficient shear connection between the slab/deck and the member. Joists and girders are assumed to be simply supported unless dynamic restraint is verified by a dynamic analysis or experiment. It is recommended that the concrete modulus be taken equal to 1.2 times that assumed in current structural standards to account for the increase in stiffness of concrete under dynamic loads. Also for determining the composite moment of inertia, the width of the concrete slab is equal to the sum of one-half the distances to the adjacent members, but each distance is not to exceed one-eighth of the span.

For the combined mode, the fundamental natural frequency can be approximated by the Dunkerly relationship

$$f_o = 0.18\sqrt{g/(\Delta_j+\Delta_g)} \tag{7}$$

where Δ_j and Δ_g are the joist and girder deflections, respectively.

The equivalent mass weight for the joist and girder panel modes are estimated from

$$W = w\,B\,L \tag{8}$$

where w is the supported weight per unit area, L is the member span and B is the effective width determined from

$$B_j = 2\,(D_s/D_j)^{1/4}\,L_j \tag{9a}$$

for the joist panel mode and from

$$B_g = 1.7\,(D_j/D_g)^{1/4}\,L_g \tag{9b}$$

for the girder panel mode, where D_s, D_j and D_g are the flexural rigidities per unit width in the slab, joist and girder directions, respectively. The following limitations and requirements apply:

1. The joist effective panel width, B_j, should not be taken greater than two-thirds of the total floor width perpendicular to the joists.
2. If the joist is connected to the girder by a single pin-type connection, the factor 1.7 in Eqn. [9b] should be reduced to 1.4. This requirement does not apply to rolled joists which are shear-connected to girder webs.
3. Where joists or girders are continuous over their supports and an adjacent span is greater than 0.7 times the joist or girder span, respectively, the effective joist or girder weight can be increased by 50%. This requirement applies to rolled sections shear-connected to girder webs.

For the combined mode, the equivalent mass weight can be approximated from

$$W = \frac{\Delta_j}{\Delta_j+\Delta_g}W_j + \frac{\Delta_g}{\Delta_j+\Delta_g}W_g \tag{10}$$

If the girder span, L_g, is less than the joist panel width, B_j, the combined mode is restricted and the system is effectively stiffened. This can be accounted for by reducing the deflections used in Eqns. (7) and (10) to

$$\Delta_g = \frac{L_g}{B_j}(\Delta_g) \tag{11}$$

where $0.5 \le L_g/B_j \le 1.0$

4. APPLICATION

Application of the proposed criterion requires careful consideration by the structural engineer. For instance, the acceleration limits for footbridges are meant for situations with many walking pedestrians and not for quiet areas like crossovers in hotel atria. For the later case it is suggested that the office floor acceleration limits be used.

Designers of footbridges are cautioned to pay particular attention to the location of the concrete slab. The first writer is aware of a situation where the designer apparently "eye-balled" his design based on previous experience with floor systems. In this case, the concrete slab was located between the beams at mid-depth (because of clearance considerations) and the footbridge vibrated at a much lower frequency and at a larger amplitude than anticipated because of the reduced transformed moment of inertia. The result was a very unhappy owner and an expensive retrofit.

Unsupported floor edges, as in many mezzanine areas, are also a special consideration because they are often lightly loaded and possess little damping. In this instance, the edge members will be made stiffer by use of the following. Where the edge member is a joist, the equivalent mass weight of the joist panel can be estimated using Eqn. (8) by replacing the coefficient 2 in Eqn. (9a) with 1. Where the edge member is a girder, the equivalent mass weight of the girder panel is the tributary weight supported by the girder. The edge panels are then combined with these orthogonal panels using Eqns. (7) and (10).

A type of light-truss joist used in North America is supported at the ends with a shoe or seat as shown in Fig. 2. This support detail affects the response of both the joist panel and the girder panel. For the joist panel, the coefficient 1.7 in Eqn. (9b) should be reduced to 1.4. For the girder panel where the concrete is separated from the concrete slab, the shoes or seats may act as Vierendal girders causing partial composite action. It is recommended that the moment of inertia of girders supporting joist seats be determined from:

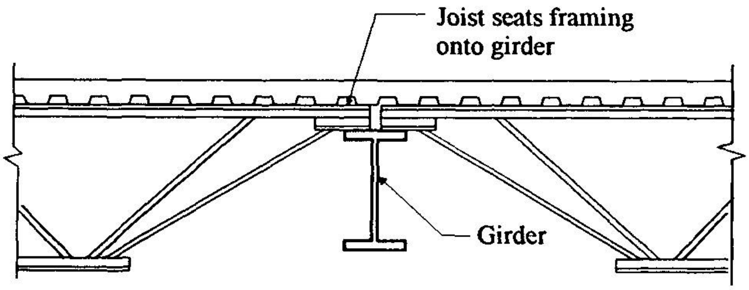


Figure 2 Light Truss Support

$$I_g = I_{nc} + (I_c - I_{nc})/2 \qquad (12a)$$

for joist seat heights 75 mm or less, and

$$I_g = I_{nc} + (I_c - I_{nc})/4 \qquad (12b)$$

for seats heights 100 mm or more, where I_{nc} and I_c are non-composite and fully composite moment of inertia, respectively.

If the bottom chord of a light truss is extended and attached to the girder, the coefficient 1.7 in Eqn. (9b) applies, since continuity is achieved. However, the fundamental frequency, f_o, does not change if the adjacent spans are approximately the same length. From Inequality (5), it is seen that an increase in the mass weight, W, results in a lower required fundamental natural frequency. This fact suggests that, if a seated joist supported floor system (as shown in Figure 2) is not satisfactory, extending the bottom chords may be an effective remedial measure.

When the natural frequency of a panel is greater than 9 hz, harmonic resonance does not occur, but walking vibration can still be annoying. Experience indicates a minimum stiffness of 1 kN per mm is required for office occupancies. To ensure satisfactory performance of office floors with frequencies greater than 9 hz, the design should use this stiffness criterion in addition to Inequality (5) when evaluating a proposed floor system.

Occasionally, a floor system will be judged particularly annoying because of what feels to be motion transverse to the supporting joists. In these situations, when the floor is impacted at one location there is a perception that a "wave" moves from the impact location in a direction transverse to the supporting joists. The first writer has observed this phenomenon and felt the "wave" 15-20 m from the impact location perhaps up to 1 second after the impact. In at least one instance, the "wave" rebounded from the exterior wall and was felt at the impact location. This phenomenon occurred in a rectangular building where the floor was free of partitions and all joists were equally spaced and of the same stiffness, including those at the column lines. The resulting motion is very annoying to occupants because the floor moves without apparent reason as the cause is not within sight or hearing. The proposed criterion does not address this phenomenon but a small change in the structural system will eliminate the problem. If one joist stiffness or spacing is changed periodically, say every third bay, the "wave" is interrupted at that location and floor motion is much less annoying.

5. CONCLUSIONS

A relatively simple criterion for the control of vibration of steel-framed floor systems for walking is proposed. Recommended values for the criterion parameters are suggested, but are expected to be improved with further experience. The proposed criterion can be applied to a number of special situations.

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Minimizing Floor Vibration Caused by Building Occupants

Minimalisation des vibrations de planchers provoquées par la foule

Minderung von menschenverursachten Deckenschwingungen

Sarah E. MOURING
Assistant Professor
United States Naval Academy
Annapolis, MD, USA

Sarah E. Mouring received her Ph.D. degree from The Johns Hopkins University, Baltimore, MD, in 1992. Currently, she is an assistant professor at the United States Naval Academy, Annapolis, MD.

Bruce R. ELLINGWOOD
Prof. Dr.
The Johns Hopkins University
Baltimore, MD, USA

Bruce R. Ellingwood received all Civil Engineering degrees at the University of Illinois in Urbana-Champaign. He was Head of Structural Engineering at the U.S. National Bureau of Standards before coming to Hopkins in 1986.

SUMMARY

One major serviceability consideration in modern buildings is excessive floor vibration due to occupant activities. Methods for accurate prediction of these vibrations and evaluation of floor systems are not readily available to the design community. An investigation is made into the characteristics of crowd-induced loads. The load characteristics are incorporated into simplified but realistic load models. Analytical procedures are developed to determine the influence of each load characteristic on the dynamic response of floor systems. Design guidelines are developed for systems subjected to crowd-induced loads.

RESUME

Les très fortes vibrations engendrées par les activités des occupants représentent l'un des aspects essentiels dans la détermination de l'aptitude au service des immeubles modernes. Il n'y a pas de méthodes servant à pronostiquer avec exactitude de telles vibrations et à évaluer le comportement des systèmes de planchers. Les auteurs examinent les caractéristiques des charges provoquées par la foule, puis les traduisent par des modèles de charge. Ils développent des méthodes analytiques pour la détermination de l'effet de chaque caractéristique de charge sur la réponse dynamique des systèmes de planchers. Ils en tirent finalement des directives de calcul pour les systèmes de dalles sous charges dues à la foule.

ZUSAMMENFASSUNG

Einer der wichtigsten Aspekte für die Gebrauchstauglichkeit moderner Gebäude sind starke, durch die Benützer verursachte Schwingungen. Methoden zu ihrer genauen Vorhersage und der Bewertung von Deckensystemen stehen der Berufswelt nicht ohne weiteres zur Verfügung. Eine Untersuchung betrifft die Charakteristiken der Einwirkung von Menschenmassen, die in einfache aber realitätsnahe Lastmodelle umgesetzt werden. Ferner werden analytische Verfahren zur Bestimmung des Einflusses jeder Kenngrösse auf die dynamische Deckenantwort bestimmt. Daraus resultieren Entwurfsrichtlinien für Deckensysteme.

1 INTRODUCTION

Floor vibrations have become a major serviceability consideration with the increasing use of high-strength, light-weight materials in modern building construction and the demand for open-space areas in office and commercial retail buildings. Floor systems in modern buildings have longer spans and are more flexible than in the past, and may have natural frequencies of vibration that fall within the range of rhythmic human activities. Floors in a number of different buildings built in the last few decades have experienced objectionable vibrations due to human activity [2,7]. Current design guidelines may not enable the structural engineer to deal with the floor vibration serviceability limit state effectively in designing floor systems. In particular, improved serviceability criteria and design guidelines need to be developed for floor systems in shopping malls, pedestrian walkways and concourses, and gymnasiums. These systems often are relatively light and are susceptible to vibration problems due to crowd-induced loading.

This paper presents the results of an investigation into characteristics of crowd-induced loads and dynamic response of floor systems [5]. These characteristics, many of which have been neglected in prior load modeling studies, include the density of the crowd, randomness of crowd movement, crowd activity, and temporal interaction between individuals. Simplified but realistic models of the crowd-induced loads are developed. Guidelines which can be used in the design and evaluation of malls, gymnasiums, and walkways are developed using these load models and dynamic analysis procedures.

2 FORCE MODELS AND DYNAMIC ANALYSIS

The starting point in developing accurate force models of human activities is the representation of the force due to an individual human footfall. Footfall force functions of several different activities, including slow walking, normal walking, brisk walking, running, and aerobics, were evaluated by Fourier analysis [1,5] (see Fig. 1). It was found that the forces could be represented by Fourier sine series with from 3 to 10 terms, depending on the pacing frequency of the individual. Using these force models, two techniques were developed to predict vibrations due to occupant-induced loads. The first technique was based on the simulation of forces due to individuals and groups of people on floor systems in the time domain and used the finite element analysis (FEA) package ABAQUS to calculate the response of the floor system due to dynamic loading. A second and simpler method involved the development of a frequency-domain solution using random vibration theory.

2.1 Time-Domain Method

The data needed to describe the activities (walking and/or running), movement, and physical make-up of the crowd include group size, individual weights, starting locations, directions of movement, coherency of movement, and pacing frequencies. The stride length, step duration, and type of footfall function for each individual are functions of the pacing frequency [5].

When simulating a group walking or running across a two-way floor system, first it was assumed that the probability of an individual entering the floor system at any location along any of the floor edges was described by a uniform probability distribution function (PDF). The starting direction of motion for each individual was given by the angle, α, with respect to the floor edge, which also was assumed to be uniformly distributed between 0 and 180°. The loading due to an individual was represented by a moving point load-time history and was multiplied by interpolation functions to determine the equivalent nodal moment- and point load-time histories for use in the dynamic FEA. After this procedure was completed for each individual in the crowd, the individual forces were shifted in time to account for randomness in the pedestrian arrival times and the nodal time histories of each individual were superposed to create nodal time histories for the group.

For a group engaging in aerobic exercise, the general approach is the same as above; however, the loading is different. One difference is that aerobic exercise is usually performed to music

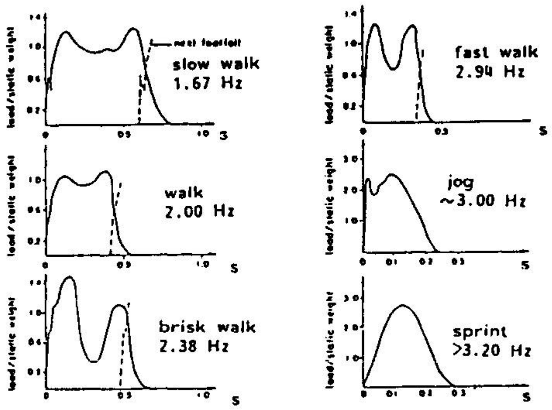

Fig. 1: Footfall Force Functions [2]

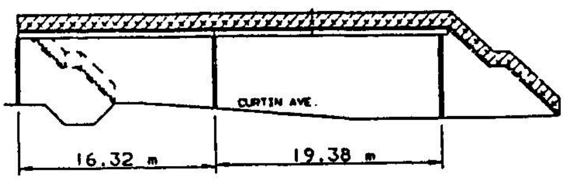

Fig. 2: Curtin Ave. Footbridge [8]

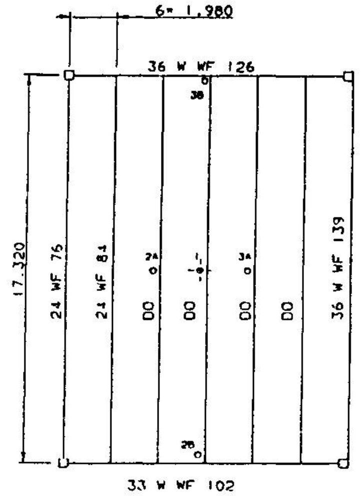

Fig. 3: Shopping Mall Floor [7]

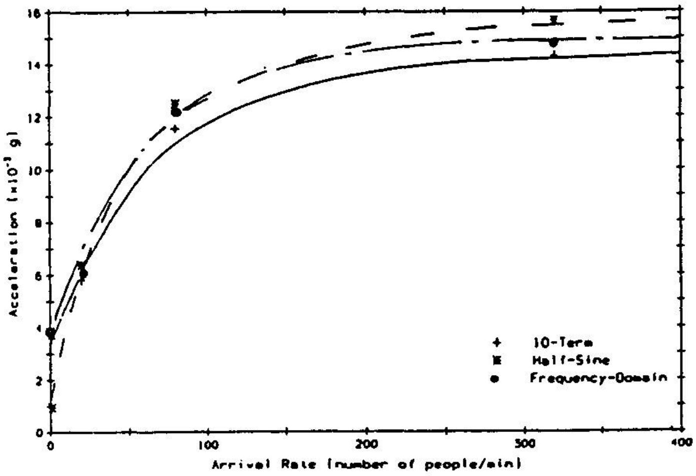

Fig. 4: Floor Accelerations: Groups Walking

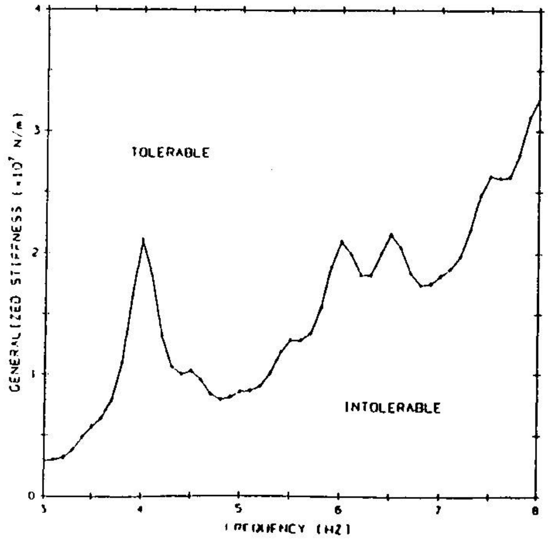

Fig. 5: Design Limits for Reference Walkway

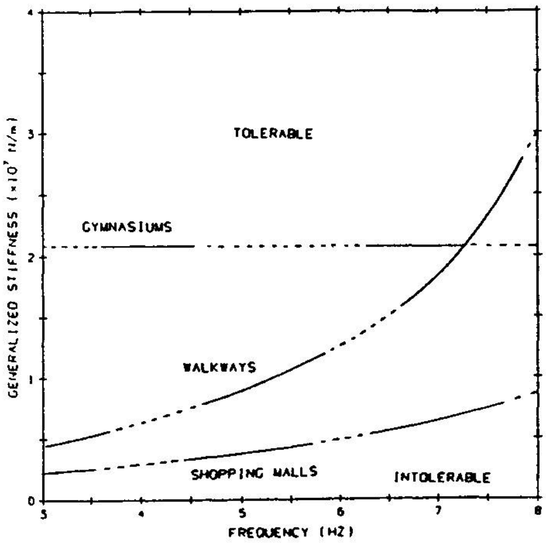

Fig. 6: Design Guidelines

in a class setting where the participants remain in place and their locations can be modeled as uniformly distributed over the floor system [5]. A second difference is that individual phase lags within a group exercising are better described by an exponential distribution due to auditory cuing from the music [3].

2.2 Frequency-Domain Method

The simulation of an individual walking or exercising was the same as described in Sec. 2.1. However, a generalized force-time history for the dominant modes was calculated for each individual instead of nodal force- and moment-time histories. This procedure was repeated for all the individuals in the crowd. The generalized force-time history, $F_g(t)$, of the crowd-induced loading then was produced by superposing the individual generalized force-time histories using the individual random phase lags. The one-sided PSD of the generalized force, $S_{f_g}(\bar{f})$, was calculated from this time history using Fast Fourier Transforms. With the determination of $S_{f_g}(\bar{f})$, the variance of the acceleration response of a floor system can be calculated in the frequency domain using standard random vibration methods [9]:

$$\sigma_a^2 = \int_0^\infty \mid H_a(\bar{f}) \mid^2 S_{f_g}(\bar{f}) d\bar{f} \tag{1}$$

where $H_a(\bar{f})$ is the system transfer function, defined by the acceleration response of a SDOF oscillator to the excitation, $\exp(i2\pi\bar{f}t)$.

3 ANALYSES OF FLOOR SYSTEMS

Several structures were analyzed as a part of a sensitivity study to test the validity of modeling assumptions made in this study and to determine those factors that most influenced the dynamic response of floors [5].

3.1 Simple Floor Systems

Two simply supported floor systems subjected to one-way, randomly phased crowd motion were analyzed. First, the significance of random pacing frequencies was investigated by selecting the pacing frequencies for the members of the crowd from a uniform PDF (uniform between 1.7 and 2.3 Hz). (See Fig. 1.) The dynamic displacements of two 16m floor systems (5 and 10Hz) subjected to a randomly paced group were approximately 15% larger than the displacements of the same systems subjected to a group pacing with the common frequency of 2.0 Hz. Second, the total peak displacement response at midspan of a floor system (16m, 5Hz) subjected to a crowd with a common pedestrian weight of 700N was found to be 7% less than the response of the same floor system subjected to a crowd with pedestrian weights normally distributed with mean 700N and standard deviation 145N. Therefore, assumptions of a common pacing frequency and pedestrian weight were made in subsequent analyses, since they do not appear to affect the response of the floor system significantly and greatly simplify the force modeling.

3.2 Footbridge

The Curtin Ave. footbridge is one of 21 footbridges in Perth, Australia used in an experimental and analytical study by Wheeler [8] of objectionable vibrations of pedestrian walkways. It is a two-span steel structure with the main span having a length of 19 m (see Fig. 2). Wheeler calculated the first and second frequencies of vibration as $f_{1_{calc}} = 3.7$ Hz and $f_{2_{calc}} = 4.3$ Hz, respectively, and the responses of the footbridge due to a person traversing the structure at pacing frequencies equal to 2.0 Hz and natural frequency (3.7 Hz). The load model used was a "half sine pedestrian model" and the weight of the test pedestrian was equal to 700N.

The fundamental frequency calculated in this study was 3.28 Hz. The second calculated frequency of 4.29 Hz could only be compared to Wheeler's calculated value of 4.3 Hz, since he did not provide the experimental frequency for the second mode in his paper. The experimental and calculated responses of the footbridge subjected to a 768N pedestrian pacing at 2.0 Hz and f_1 are compared in Table 1. It should be noted that the first mode represented 92% of the total displacement response. The responses calculated by Wheeler did not compare as well to the experimental results, most likely because of his approximations in the force model.

3.3 Floor System in a Shopping Mall

The floor system analyzed is located on the second story of a mall in Canada, and reportedly had noticeable floor vibration due to pedestrian-induced loads [7]. The floor is of composite construction (see Fig. 3). In-situ measurements of accelerations were taken at designated locations noted on Fig. 3. The floor system was excited by a 935N person performing heel-drop impacts at each accelerometer location, and by the same person walking past and between specific accelerometers at a normal pacing frequency of approximately 2.0 Hz. The fundamental frequency measured was $f_{1_{exp}} = 4.0$ Hz, while the fundamental damping value determined by the log decrement method was 3.3%.

The fundamental frequency of the floor system was calculated in this study to be 4.26 Hz. A comparison of predicted and measured accelerations is given in Table 2. The first mode represented 96% of the total acceleration response. A sensitivity study on group activity revealed that [5]:

- The assumption in a floor system evaluation that an individual treads in place at midspan rather than walks across the floor overestimates the calculated peak acceleration by 28%.
- The floor acceleration and the pedestrian arrival rate are related by a factor $\sqrt{N}$, where N is the number of randomly phased people walking on the floor at a given time (see Fig. 4).
- The half-sine shape approximation of the individual footfall function is adequate for group loading but not for individual loading (see Fig. 4).
- The total acceleration of the floor system subjected to a group of people exercising with exponentially distributed phase lags is less than 50% of the response when the same group exercises completely in phase.

The accelerations calculated by the frequency-domain method considering only the first mode compared very well to the responses calculated by the time-domain method for both groups exercising and groups walking. The time-domain and frequency-domain response values due to groups of individuals with a common body weight (935N), common walking frequency (2.0 Hz), and random individual arrival times were within 15% of each other (see Fig. 4).

4 GENERAL DESIGN GUIDELINES

Serviceability of floors traditionally has been addressed by requiring that the deflection of the floor system due to live load be less than some fraction, typically 1/360, of the span length. Other important factors governing dynamic response, including the mass and damping of the floor system, are not reflected in this requirement. It has been suggested recently that the static deflection of the floor system under a 2KN force applied at midspan should be less than 1mm to provide sufficient static stiffness against walking vibration [4]. Limiting absolute static deflection is tantamount to limiting the fundamental frequency, but does not directly deal with the dynamic component of the load. It also has been suggested that excessive vibrations often can be avoided by designing floor systems to have fundamental frequencies above a certain value (typically about 8 Hz) [1,6]. However, this alternative may not always be economical. Designers ought to have other methods that deal with directly the dynamic nature of the loads and are relatively easy to implement in practice.

4.1 Development of General Design Guidelines for Walkways

The development of design guidelines for walkways was based on the results summarized in Sec. 3 [5]. The first step was to perform a FEA of a walkway subjected to crowd loading. A simply supported, one-span reference system with length L (18.36m) and damping value of 3% was designed to specification and modeled using finite elements. It was assumed that the maximum arrival rate of people was 120 people per minute, that each pedestrian weighed 700N, had a common pacing frequency (2.0 Hz) and forward speed, and a random arrival time. The loading was calculated using the method described in Section 2.1. The natural frequency, generalized mass, and acceleration-time history at the center node were calculated for the first mode. The power spectral density (PSD) of acceleration, $S_a(\bar{f})$, for the first mode was calculated from the acceleration-time history using Fast Fourier Transforms. After calculating the system transfer function, the PSD of the generalized force at the center node for the first mode was determined by:

$$S_f(\bar{f}) = \frac{S_a(\bar{f})}{\mid H_{a_{ref}}(\bar{f}) \mid^2} \tag{2}$$

in which $H_{a_{ref}}(\bar{f})$=system transfer function for the reference floor.

Any simply supported floor systems of length L subjected to the same crowd behavior has a PSD of the generalized force of the first mode equal to $S_f(\bar{f})$. If acceleration, $a(t)$, can be assumed to be a stationary process [5,9], then the rms acceleration can be calculated from Eqns. 1 and 2. The peak acceleration is related to the rms acceleration by a peak factor, found to be equal to approximately 3.0 for the floor systems considered herein by inspecting outputs from the time-domain analyses [5]. Therefore, the peak acceleration is calculated by:

$$a_p = 3.0\sqrt{\int_0^{\infty} \mid H_a(\bar{f}) \mid^2 S_f(\bar{f}) d\bar{f}} \tag{3}$$

in which $H_a(\bar{f})$=system transfer function for the floor considered.

A design chart for walkways can be developed for the reference system mentioned above. The recommended peak acceleration limit for walking vibration on walkways is 5% g [2]. By varying the fundamental frequency, values of the generalized stiffness can be determined for which the peak acceleration is 5% g. These specific systems also must satisfy strength and static deflection requirements. A curve was developed in this manner which identifies systems as being either tolerable or intolerable (see Fig. 5). Figure 6, where a smooth curve has been fitted, is proposed for design purposes. The dotted lines in Fig. 6 represent the zones of resonance and should be avoided.

Figure 6 cannot be used directly with simply supported floor systems with span lengths other than L, since a floor system of length L/N has 1/N times the number of pedestrians on the system at one time due to the constant pedestrian arrival rate assumed for all systems. However, the generalized stiffness in Fig. 6 can be scaled by the factor $\sqrt{N}$ for use with other span lengths. Thus if the floor system has span length, l, the generalized stiffness is scaled by $\sqrt{L/l}$ where L=18.36m and K_1^*=scaled generalized stiffness. Finally, the point (f_1,K_1^*) is plotted to see if it lies in the tolerable or intolerable area of the chart. Figure 6 also can be used for floor systems of length L with different arrival rates by scaling the generalized stiffness by the square-root of the ratio of 120 people/min to the given arrival rate.

4.2 Evaluations of Walkways

Six simply supported, one-span walkways were evaluated after first being designed to meet strength and traditional (static) serviceability requirements. By plotting the scaled generalized stiffness, K_1^*, and fundamental frequency, f_1, on the design chart, only Walkway 4 was found to have tolerable accelerations (see column 7 of Table 3). To validate the proposed design chart, these results are checked with results calculated using simulated force-time histories and FEA to compute dynamic responses (see column 8 of Table 3). In every case, peak accelerations above 5%g corresponded to intolerable ratings given in column 7 of Table 3. Therefore, the design chart identified those walkways that had unacceptable vibrations due to heavy crowd loading.

Other serviceability criteria considered included limiting the maximum deflection under a uniformly distributed design live load to be less than L/360, and limiting maximum deflection under 2KN point load located at midspan be less than 1mm [4]. Deflections under a 2.9kPa uniformly distributed nominal live load and under a 2KN point load are given in columns 9 and 10 of Table 3. Both criteria were satisfied for all six walkways; however, Walkway 4 was the only system to have a tolerable rating when dynamic response was considered. Therefore, walkway designs that satisfy the static deflection criteria may vibrate objectionably when heavily trafficked.

4.3 Summary of Design Guidelines for Other Occupancies

Design guidelines for shopping malls and gymnasiums also were developed as part of this study (see Fig. 6) [5]. However, the zones of resonance for the gymnasium guidelines are very wide. Therefore, there is only a relatively small range of fundamental frequencies in which a gymnasium floor can be designed using the proposed design guidelines. It is recommended that floor systems in buildings where exercise classes are regularly scheduled be designed to have fundamental frequencies above 10 Hz. This value is greater than the third harmonic of the forcing frequency most likely to be encountered on the floor.

5 CONCLUSIONS

General design guidelines were developed to evaluate simply supported floor systems where the occupants walk or exercise in groups. These may be used as screening tools for floor systems with different boundary conditions. The design guidelines are limited to a few common activities and building occupancies. Other guidelines need to be developed for floor systems subjected to different activities.

6 ACKNOWLEDGEMENTS

Computional support from the Pittsburgh Supercomputer Center through Grant No. MSM910013P is greatly appreciated. Assistance in this study by J.E. Wheeler in providing working drawings and other documentation also is appreciated.

	Total Peak Response (mm)		
	Calculated (Wheeler)	Experimental (Wheeler)	This study
Pacing = 2.0 Hz	0.70	0.60	0.59
Pacing $= f_1$	9.9	7.5	7.0

Table 1: Comparison of Results of Curtin Ave. Footbridge

Transducer Setup	Type of Test	Peak Acceleration (x10e-3g) at Transducer Location					
		Experimental			Calculated		
		1	2	3	1	2	3
A	Heel Impact at (1)	7.0	8.7	9.2	10.2	8.4	9.5
A	Walking Past (1,2,3)	2.6	3.0	3.2	3.6	3.7	3.0
B	Heel Impact at (1)	7.7	3.1	1.5	10.2	3.6	3.1
B	Walking Past (1,2,3)	2.8	2.0	1.4	3.7	1.3	1.1

Table 2: Comparison of Results of Shopping Mall Floor

Walkway (1)	L (m) (2)	f_1 (Hz) (3)	M_1 (kg) (4)	K_1 (N/m) (5)	K_1^* (N/m) (6)	Chart Eval. (7)	$a_{P,calc}$ (%g) (8)	L/Δ_{LL} (9)	$\Delta_{P=2Kn}$ (mm) (10)
1	18.36	3.41	4.66e3	2.14e6	2.14e6	INTOL.	11.0	595	0.94
2	18.36	4.95	5.68e3	5.49e6	5.49e6	INTOL.	7.5	1529	0.36
3	18.36	5.70	6.35e3	8.13e6	8.13e6	INTOL.	8.4	2260	0.25
4	18.36	7.46	1.06e4	2.33e7	2.33e7	TOL.	5.0	6397	0.09
5	13.77	6.59	4.21e3	7.20e6	8.31e6	INTOL.	12.0	1978	0.28
6	9.16	7.10	2.37e3	4.75e6	6.71e6	INTOL.	14.0	1295	0.43

Table 3: Comparison of Methods to Evaluate Footbridges

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Measurement of Floor Vibration

Mesures des vibrations de planchers

Messung von Deckenschwingungen

Naoki UCHIDA
Struct. Engineer
Nikken Sekkei Ltd
Osaka, Japan

Haruo TSUKAGOSHI
Struct. Engineer
Nikken Sekkei Ltd
Osaka, Japan

Tsukasa AOYAGI
Struct. Engineer
Nikken Sekkei Ltd
Osaka, Japan

Koichi TAKANASHI
Professor
Univ. of Tokyo
Tokyo, Japan

SUMMARY

Recently, the Architectural Institute of Japan compiled comprehensive guidelines for evaluating the habitability of buildings with respect to vertical floor vibration by making reference to exhaustive related research findings in the past. The authors, having conducted measurements of vertical floor vibration of a large number of buildings over many years, examine the reasonableness of the AIJ's evaluation guidelines by using related field data and describe a practical design concept that ensures sound floor beams giving special attention to the beams supporting office floors.

RESUME

La Société des Architectes du Japon (AIJ) a dernièrement établi des directives très complètes pour l'évaluation de l'habitabilité des immeubles en ce qui concerne les vibrations verticales des planchers, sur la base de l'ensemble des résultats de travaux menés par le passé. Les auteurs, qui ont procédé pendant de longues années à des mesures de vibrations verticales des planchers d'immeubles, étudient le bien-fondé des directives d'évaluation de l'AIJ en faisant appel aux données disponibles dans des domaines connexes et ils décrivent les concepts qui, dans la pratique, assurent la robustesse des poutres de plancher, en s'intéressant tout particulièrement aux poutres soutenant les planchers des bâtiments commerciaux.

ZUSAMMENFASSUNG

Jüngst hat das Architectural Institute of Japan umfassende Richtlinien für die Bewertung der Bewohnbarkeit von Gebäuden hinsichtlich vertikaler Deckenschwingungen erstellt, indem auf erschöpfende Forschungsergebnisse der Vergangenheit Bezug genommen wurde. Die Autoren, die Messungen der vertikalen Deckenschwingungen in einer grossen Anzahl von Gebäuden über viele Jahre durchgeführt haben, untersuchen die Verhältnismässigkeit der AIJ-Bewertungsrichtlinien durch Verwendung einschlägiger Felddaten und beschreiben ein praktisches Bemessungskonzept, das insbesondere für Bürogebäude ausreichende Deckenträgerquerschnitte liefert.

1. MEASUREMENTS OF FLOOR VIBRATION

In Japan, the achievements accumulated in many years through exhaustive researchers on the structural design method and the habitability evaluation method, both related to serviceability of buildings, were compiled in comprehensive form results. The fruit of such compiling efforts can be seen in the two books published lately by Architectural Institute of Japan (AIJ), which has always played a leading role in setting out the nation's building design criteria. One related to the former is Standard for Limit State Design of Steel Structures (draft) published in 1990 while the other related to the latter is Guidelines for the Evaluation of Habitability to Building Vibration published in 1991.

As a new design method alternative to the conventional allowable stress design, the limit state design (LSD) employs "limit state design method" which prescribes that structures be designed according to the design criteria based on the limit state of steel structures. In this method, two types of limit state are established: one is the limit state as to the structural safety of buildings and the other is the limit state as to the serviceability and habitability of buildings.

Of these two, the serviceability limit state design requires that the following three principles be observed as design basics: 1. design considering the limit strength of a structure during use; 2. design considering the limit deformation of a structure during use; and 3. design considering the lateral sway and vibration of floors due to floor vibrations and wind force and also adverse vibrations due to any other causes.

Of the three principles mentioned above, the one concerned with the design consideration for serviceability and habitability of steel buildings mentioned in Item 3 prescribes that "the design shall provide means to deal with floor vibrations and lateral sways of a building as necessary. In such design, proper working loads and acceptable levels of vibrations, lateral sways, etc. shall be set out based on the required service and functional performance of the building." The basic concept has had to be expressed in this way because the performances required of buildings vary with their intended purposes, sizes, shapes, etc. and therefore are not amenable to a uniform definition.

The Guideline for Evaluation published in 1991 comprises two kinds of criteria, i.e., one for the vertical vibration of the floor and the other for the lateral vibration of the building. While the habitability in a broad sense comprises such factors as safety, functions, sanitation/hygiene and comfort, the guidelines primarily aims at securing daily living comfort which is habitability in a narrow sense, and in order to allow as much design freedom as possible, there are intended to provide a general guide for performance evaluation rather than to define acceptability strictly.

As for the evaluation of habitability related to vertical vibration of building floors, the guidelines provide the criteria for performance evaluations in terms of displacement amplitude and acceleration amplitude for three types of vibrations which have different characteristics and for different types of buildings classified by their intended uses. As for the intended purposes of buildings, the following three categories are considered: living rooms and bedrooms in residential buildings; conference and drawing rooms of office buildings; and general office spaces of office buildings. The guidelines also prescribe that responses to floor vibrations be evaluated by three kinds of methods which include predictions analyses and actual measurements. As for the vibration-related criteria set out by AIJ, the relationship between vibration frequencies and displacement amplitudes were specified in "Standard value for the building structural design to present vibration-induced damage (plan)" deliberated in 1959, and the criteria contained therein had been left intact for about 30 years until they were replaced by the aforesaid new criteria. As for the restriction of beam deformation, AIJ's Design Standard for Steel Structures published in 1970 specified that the displacement under the total load should not exceed 1/300 of the span. This means that buildings in Japan kept on becoming greater in span and height while no up-to-date criteria concerning the serviceability of building floors were established. In the meantime, measurements as well as theoretical and experimental researchers related to this subject were carried out by a number of researchers and practicing engineers, and this paved the way to the compilation of the aforesaid two AIJ Standards.

The authors, too, accumulated substantial measurement data on the vertical vibrations of floors of many buildings intended for various purposes through their more-than-thirty year involvements in the design of such buildings. In this report, some of such measurement data will be presented by limiting the data to those on the office buildings where human footfalls are thought to be a principal cause of floor vibrations. Following this, practical means to deal with such vibrations will be introduced from a viewpoint of structural design and the applicable performance evaluation criteria will be examined. Lastly, some consideration given to the design be the authors to prevent hard-to-predict vibration nuisances will be described.

2. STRUCTURAL DESIGN OF FLOOR BEAMS

As preparatory means to design sound floor beams free of adverse vibrations, the authors have been accumulating measured floor vibration data on a variety of buildings. Since the acceptable levels of floor vibrations are believed to vary with the intended purposes of buildings, the authors will introduce in this paper a number of cases limiting the floor beams to those supporting office floors where vibrations are mainly created by human footfalls. Floor beams must above anything else be designed to have functions to support working dead loads and live loads and transfer them to the foundations by way of columns. In areas like Japan where seismic force is a predominant factor, floor beams are often designed to be in the form of girders which serve as elements in aseismic framework; hence, in designing such beams, not only their capacity to carry the lateral force due to an earthquake but also their bending rigidity which affect the lateral rigidity of a building structure must be considered.

Therefore, cross sectional areas of floor beams with relatively short spans are governed by the bearing capacity and rigidity required by aseismic design, and in many of such cases the rigidity required of the floor beams to resist vertical floor vibrations may consequently be regraded as negligible.

For this reason, buildings having relatively long spans accounted for the majority the buildings whose floor vibrations were measured by the authors. Consequently, most of the buildings in which floor vibrations were measured by the authors were steel structures having composite beams except a small number of buildings which had prestressed concrete floor beams. In these buildings, the steel beams were in most cases connected with reinforced concrete slabs by means of shear connectors to increase rigidity. Headed studs were generally used as shear connectors. The design method for this type of composite beams was formulated by AIJ as design guidelines in 1975. Since steel beams and concrete slabs jointly resist bending moments, composite beams display higher load-carrying capacity and rigidity than steel beams acting alone.

Fig. 1 shows a typical composite beam. Fig. 2 shows moment inertia, which are the indices of bending rigidity of members, for various sections of H-shaped steel beams and comparable composite beams. The figure is intended to show, by using slab thicknesses and effective widths of composite beams as parameters, to what extend moment inertia can be improved by adopting composite beams. This remarkable increase in bending rigidity as shown in the figure testifies clearly that composite beams are effective for preventing nuisances caused by vertical floor vibrations.

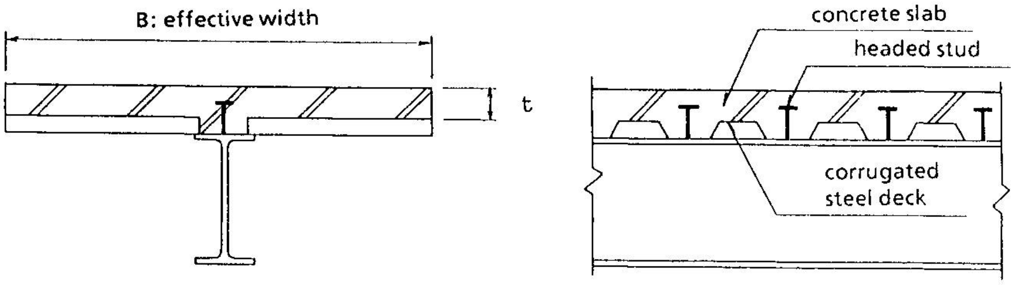

Fig.1 Composite beam

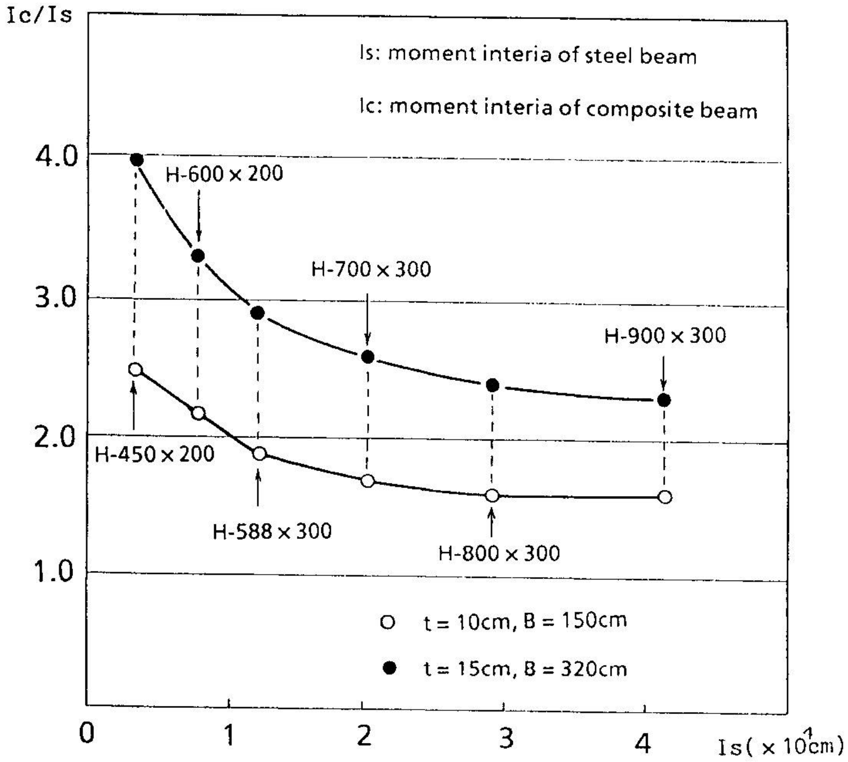

Fig.2 Increase in flectial rigidity of composite beam

3. MEASUREMENT RESULTS AND EVALUATIONS

For the purpose of measurements, the authors caused the floors to vibrate by using one of the following three methods: 1) dropping of a sand-bag, 2) forced vibration by means of an oscilator, and 3) human footfalls. Fig. 3 shows the measured frequencies of vibrations caused by Method 1) in which a bag containing compacted sand weighing 30kg was dropped by gravity to the floor to cause vibrations when the basic properties and damping constants have to be measured in order to study beam-floor interactions under vibration. A formula indicting the relationship between the vibration frequencies and the deflections due to dead loads is included in the figure.

In Fig. 4, the frequencies thus obtained are compared with the computed frequencies.

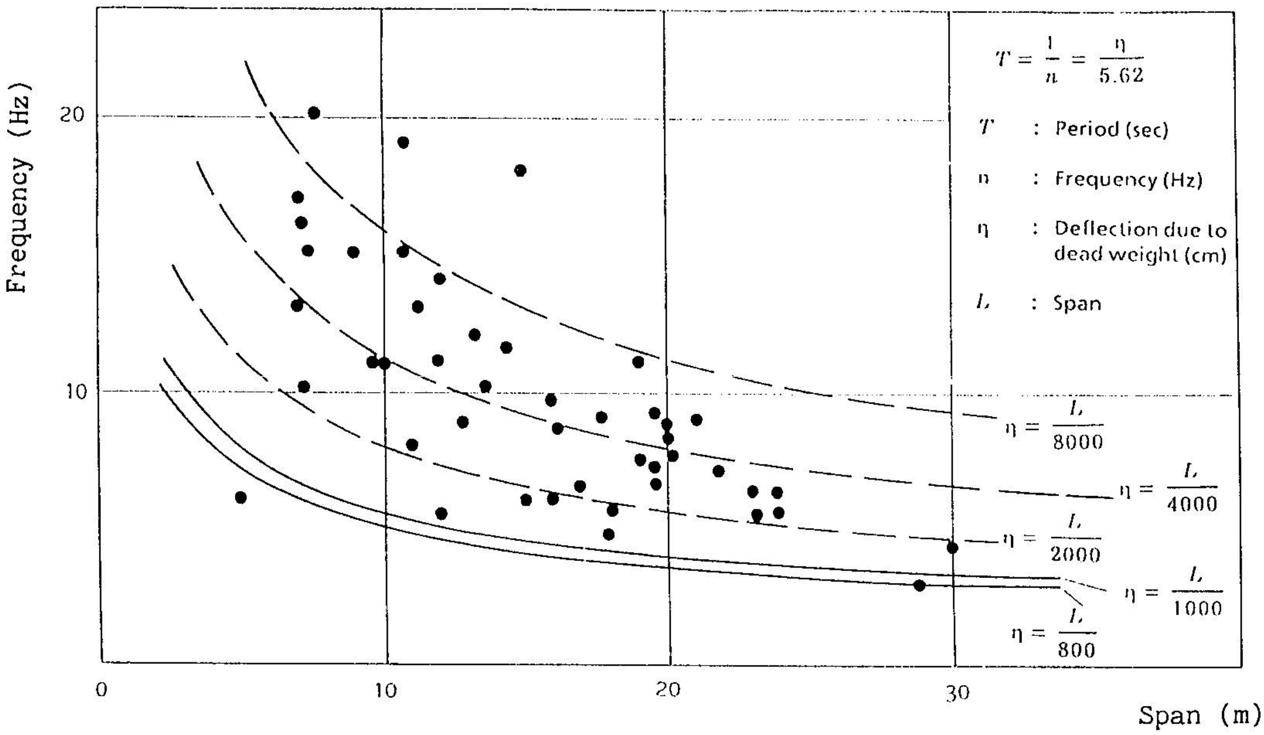

Fig.3 Measurement frequency VS. span

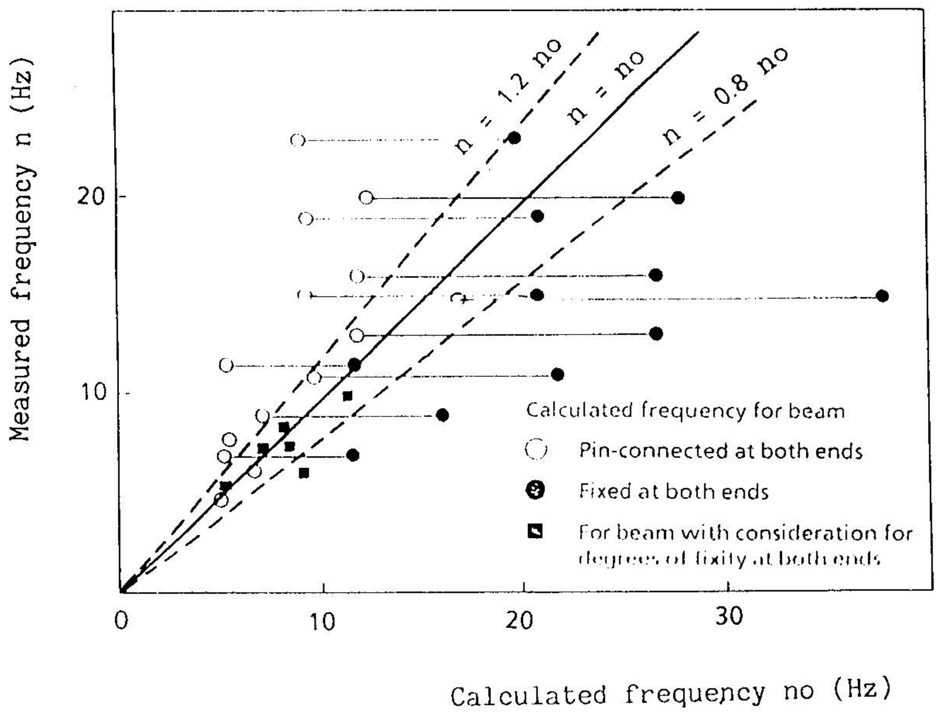

Fig.4 Frequency of floor beam

Most of these floor beams were rigidly connected to the columns to form a rigid frame; however, good agreement was seen between the measured and the computed values in case the conditions of end fixty were accurately evaluated. Where it was difficult to accurately determine the conditions of end fixity, the computed frequencies were given for both cases, i.e., for hinged connection and fixed connection, and the measured values were found to be somewhere between the computed values for these two cases.

Fig. 5 shows the relationship between the measured dynamic deflections of the floor beams when the floor supported by them was subjected to footfalls given by two persons and the vibration frequencies. This experiment was conducted to simulate a case where the floor supported by such beams was used as office space. Although the results obtained were rather erratic being influenced by the walking paces, etc. of the persons, all the plotted values, which were obtained by averaging the dynamic deflections measured several times, may be taken as data that verify sound floor beams reasonably free from adverse vibrations. The standard line V-5 as set out in AIJ Recommendation 1991 for the floor beams supporting a floor to be used as office space and the recommended line proposed by the authors at IABSE WCII workshop 1989 are indicated in the figure on a comparative basis. Both the proposed lines embrace the upper limits of the measured values obtained thus far be the authors and they agree with each other almost completely; therefore, they may be considered good formulas that serves practical purposes well under the present circumstances.

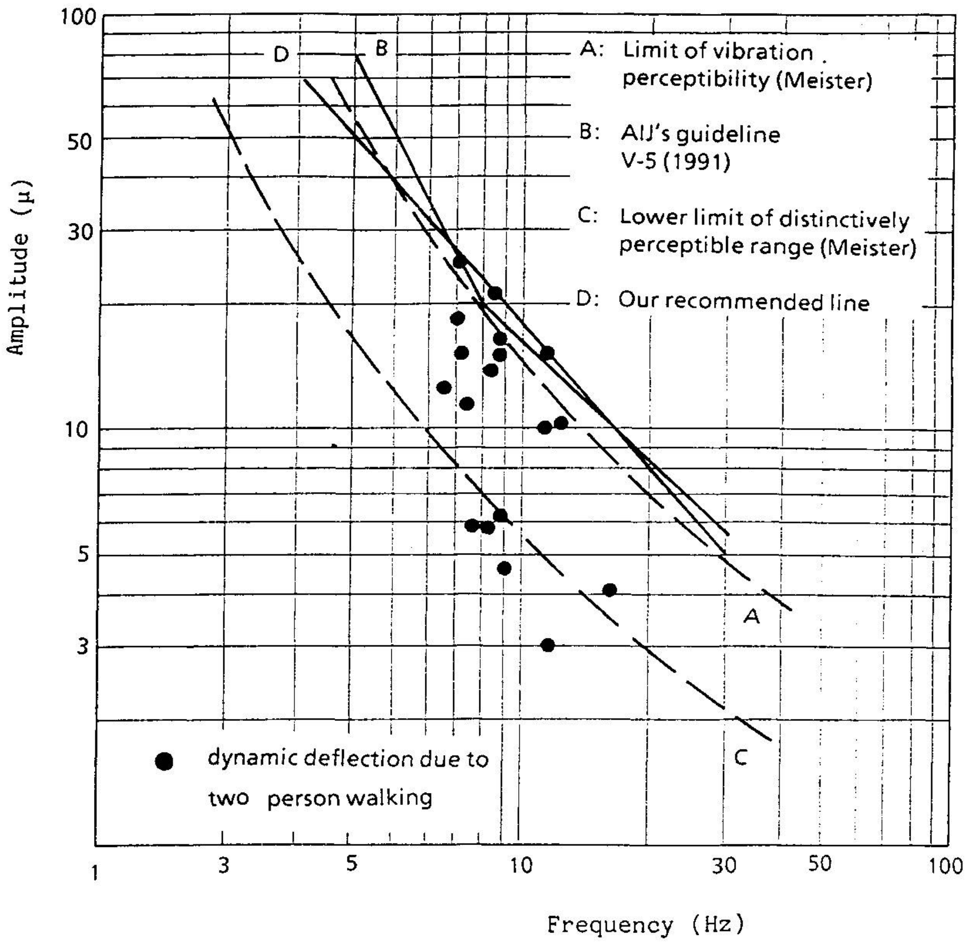

Fig.5 Measured dynamic deflection due to person walking in office building

In connection with the method of calculating dynamic deflections, dynamic deflections of beams due to lateral impacts as obtained by Timoshenko's formula given below and the comparable measured values are shown in Fig. 6.

$$\delta_d = \delta_{st} + \sqrt{\delta_{st}^2 + 2h\delta_{st}\frac{1}{1 + \alpha\cdot\frac{W_1}{W}}} \qquad (1)$$

where,

δst : static deflection of the beam caused by a falling object

δd : dynamic deflection due to impact

h : falling height

W1 : total weight of floor beam

W : weight of falling object

α : 17/35 (for simple beam supported at both ends)

The value of α indicated above is the one obtained for a beam simple-supported at both ends. For a beam fixed at both ends, the applicable value is 13/35.

Further, the impact caused by footfalls of two persons was obtained from the test data available in Japan and also from the results of researchers based on the law of conservation of momentum. Namely, predicated on a conclusion that the impact caused by the footfalls of one person is equal to that caused by a object weighing 3kg dropped by gravity from a height of 5cm, conversion was made to a case of the footfalls of two persons, and W=6kgf and h=5cm were obtained from Fig. 6.

While the measured values were generally lower than the theoretical values thus obtained, in some cases the former turned out about 1.5 times the latter.

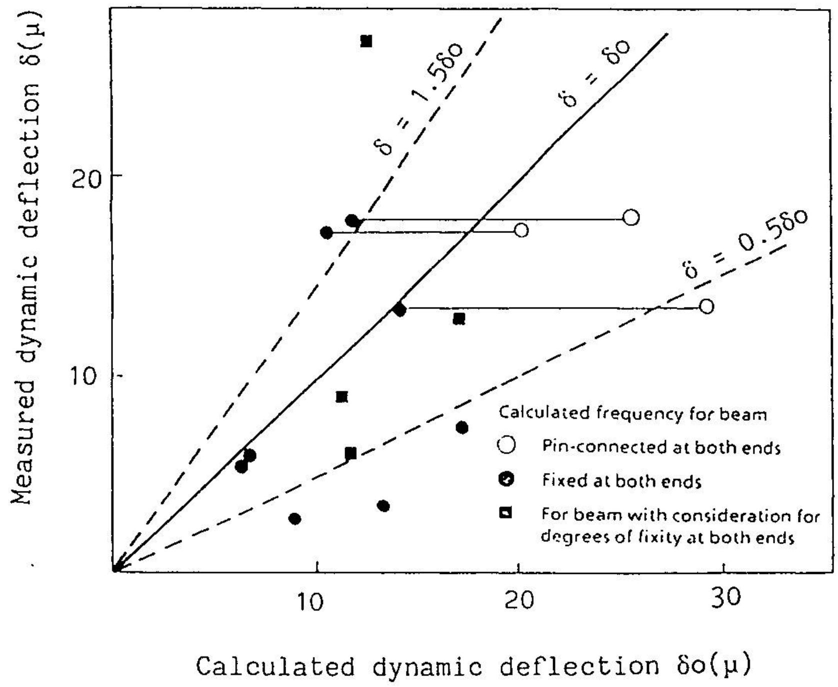

Fig.6 Dynamic deflection due to two person walking

4. CONCLUSIONS

The results of studies described above indicate the typical guidelines for office buildings given in AIJ's Recommendation 1991 correspond very well with the measured results and therefore are considered adequate. Further, under the present technological circumstances, it is proposed that the following be considered when studies as to vertical vibrations of floor beams supporting office spaces are to be made at the design stage.

1) Vibration frequency of floor beams should be computed. For computations, the fixty at beam ends should be defined as correctly as possible.
2) Dynamic deflection of floor beams due to human footfalls should be computed by Formula (1).
3) Evaluation should be made according to Standard Line V5 (AIJ Recommendations 1991) shown in Fig. 5.
4) Also, from Fig. 3 it may be considered reasonable for practical purpose to restrict the deflection of floor beams due to their deadweight to below 1/800~1/1,000 of their spans.

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Evaluation of Habitability under Building Floor Vibration

Evaluation des vibrations d'un bâtiment habitable

Bewertung der Bewohnbarkeit bei Deckenschwingungen

Hiroshi KUSHIDA
Dr. Eng.
Takenaka Corp.
Tokyo, Japan

Takeshi GOTO
Prof.
Hosei Univ.
Tokyo, Japan

Hidenori ONO
Prof.
Tokyo Inst. of Technology
Tokyo, Japan

Chikaaki UEDA
Dr. Eng.
Kumagei Corp.
Tokyo, Japan

Katuhisa OHKUMA
Eng.
Oobayashi Corp.
Tokyo, Japan

Yutaka YOKOYAMA
Prof. Dr.
Nagoya Inst. of Technology
Nagoya, Japan

SUMMARY

AIJ guidelines (Architectural Institute of Japan) are applied to test the vibrations which occur on floors in buildings. The test is done to maintain the habitability of the building. By comparing the past research, six test curves for continuous vibrations and impact vibrations were prescribed to secure the efficiency of a floor subject to vibrations. The test curves were verified by the field data obtained from a floor motion test. Further, AIJ guidelines prescribe both an analytical method for the test of the vibration efficiency of the floor and the response test method made through experiments.

RESUME

Les directives AIJ (Société des Architectes du Japon) règlent les essais de vibrations à réaliser dans des bâtiments. L'essai doit permettre de s'assurer que le bâtiment est habitable. Sur la base de recherches antérieures, six courbes pour des vibrations continues et des vibrations sous l'effet de chocs sont appliquées pour contrôler l'efficacité d'un étage exposé à des vibrations. Les courbes ont été vérifiées avec les données obtenues à partir de l'essai d'un étage en mouvement. De plus les directives AIJ demandent à la fois une méthode analytique pour l'essai d'efficacité de l'étage et la réponse d'essais expérimentaux.

ZUSAMMENFASSUNG

Vom Japanischen Architektur-Institut (AIJ) wurden Richtlinien aufgestellt, um die Schwingungen von Geschossdecken zu bewerten und die Bewohnbarkeit von Gebäuden zu gewährleisten. Aufgrund bestehender Vergleichsdaten wurden für dauernd und intermittierend auftretende Schwingungen sechs Bewertungskurven aufgestellt, mit denen die Schwingungsanfälligkeit von Geschossdecken beurteilt werden kann. Diese Kurven wurden anhand von Testdaten schwingender Decken verifiziert. Für den Nachweis des Schwingungswiderstandes sehen die AIJ-Richtlinien eine analytische Methode und Versuche vor.

1. PARAMETER OF APPLICATION OF AIJ RECOMMENDATIONS

The guidelines of AIJ are applied to the evaluation of the vertical vibration which occurs in a building for the purpose of maintaining a high level of habitability. The evaluated floors are structural floors which will be used as residential spaces, office areas, and for other similar purposes. Floating floors and double decks are not taken into consideration for the evaluation made through the application of the guidelines of AIJ. The vibration to be evaluated is a vibration acting on a building in a vertical direction to the floor's surface. Under donsideration of the actual state of the floor vibration, the natural frequency of the floor is set within an area of 3 ~ 30HZ. The evaluation of the vibration is carried out through the verification of the frequency, amplitude, and damping ratio, all of which can be obtained from the response wave of the floor with the evaluation curves. Furthermore, the floor response wave can be gained from the condition of excitation which is assumed to be the origin of the tremor that is felt under normal conditions of floor use.

2. GUIDELINES FOR THE EVALUATION OF HABITABILITY

The AIJ guidelines for the habitability for floor vibrations evaluation are shown in Fig. 1. The evaluation curves are composed of six curves on the basis of the threshold value for the sensibility which is found in the curve of V-1.5. The standard for the evaluation of habitability is classified into the following three types in accordance with the differences of the vibration behavior.
Classification 1: Habitability evaluation for the floor which is subject to a continuous vibration or a vibration which is repeated intermittenly. : V-5 or less.
Classification 2: Habitability evaluation for the floor with low damping which is subject to an impact vibration. (Damping ratio is 3% or less): V-10 or less.

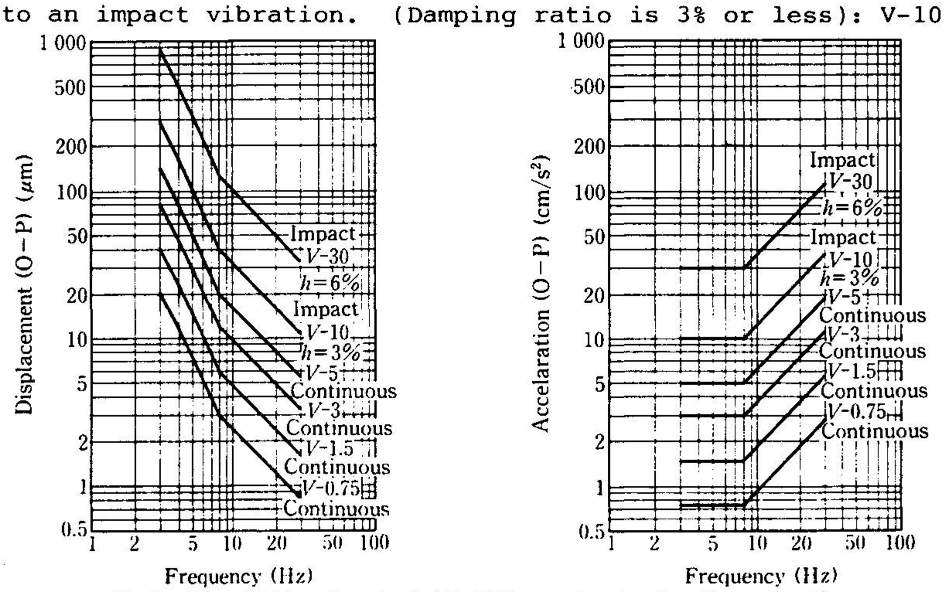

Fig.1 AIJ guidelines for the habitability evaluation for floor vibration.

Table 1:Classification of the habitability building floor vibration

Classification Bldg. Rm.	Rank	Classifi.1 Rank I	Rank II	Rank III	Classifi.2 Rank III	Classifi.3 Rank III
Resi-dential	Living.rm. & bed.rm.	v-0.75	v-1.5	v-3	v-5	v-10
Office	Conference.rm. Guest.rm.	v-1.5	v-3	v-5	v-10	v-30
	Office.rm.	v-3	v-5	v-5 Approx.	v-10 Approx.	v-30 Approx.

Note: The "Rank" represents the habitability grade, with Rank II being a general average. Rank I is such level that habitability index is recommended to be smaller than this, and Rank III not to be larger than this.

Classification 3: Habitability evaluation for the floor which possesses high damping and is subject to an impact vibration. (Damping ratio is approximately 3-6%): V-30 or less.

Table 1 shows the classification for the habitability evaluation against vibrations and that for the efficiency evaluation of the building according to the manner in which it is used. The displacement amplitude of d, which corresponds to an arbitrary frequency of f in the evaluation curves shown in Fig. 1, can be obtained through the use of the following equation.

$$d = e^{\frac{c - a \cdot \log_e(f)}{b}}$$

where f: Frequency (HZ), d: Displacement amplitude (μ m)
The value for a, b, and c shall be taken from the coefficients indicated in Table-2. The relationship between the frequency and the acceleration (α) is obtained from the equation of $\alpha = d \times (2\pi f)^2$

Table 2: The coefficients value for evaluation curves

Frequency / Evaluation curve	$3 \leqq f \leqq 8$ Hz			$8 \leqq f \leqq 30$ Hz		
	a	b	c	a	b	c
V-30	2	1	8.94	1.265	1.316	8.92
V-10	2	1	7.84	1.265	1.316	7.47
V- 5	2	1	7.14	1.265	1.316	6.56
V- 3	2	1	6.64	1.265	1,316	5.88
V- 1.5	2	1	5.94	1.265	1.316	4.97
V-0.75	2	1	5.25	1.265	1.316	4.06

3. BACKGROUND FOR THE ESTABLISHMENT OF VIBRATION EVALUATION CRITERIA

In 1959, the "standard value for the building structural design to prevent vibration-induced damage (plan)" was deliberated on by AIJ. (Fig.-3). This standard value was established for the influence of vibrations caused by the facilities and equipment present in a building. The standard value is indicated by the B-curve, which was drawn through referring to the research which was carried out by Meister on the various vibrations which a human being can perceive. Moreover, as representative evaluation criteria for floor vibrations, there are such standards and guidelines as exist below.

3.1 GSA VIBRATION EVALUATION CRITERION Note 1

For this evaluation, the following equation which uses the frequency and displacement amplitude damping ratio based on the study made by Wiss & Parmelee as a parameter. Both of them are well known for the research on transient vibrations.

$$R = 5.08\,(f\,Ao/h^{0.217})^{0.205}$$

where R: Vibration sensibility rank, f: Frequency (HZ), Ao: Displacement amplitude (in), h: Damping ratio

3.2 CSA VIBRATION EVALUATION CRITERION Note 2

This evaluation criterion was established for the continuous vibration and the impact vibration on the basis of the standard made through the study by Allen & Rainer.
The evaluation of the impact vibration is made through the use of the preliminary amplitude, frequency and the damping ratio, all of which are caused by the pounding of feet on the floor surface. The vibration, which possesses the evaluation value of 3, 10 and 30 times the standard value for the continuous vibration, is evaluated as the impact vibration with the damping ratio of 3%, 6%, and 12%.

3.3 ISO 2631/2 GUIDELINE FOR THE EVALUATION OF HUMAN EXPOSURE TO WHOLE BODY VIBRATION

This guideline is applied to the evaluation of the reaction of the human body to vibrations which can be perceived within a frequency area of 1-80HZ. The evaluation coefficient is determined in accordance with the manner in which the floor is used.

3.4 MODIFIED MEISTER CURVE

The Meister curve is also called the Lenzen curve. Lenzen reformed the Meister curve due to the fact that a curve which is almost 10 times that of the Meister curve corresponds quite well to the curve which is used for the evaluation of the vibration caused by the foot steps of a single person. From the results of the aforementioned studies, it is found that the continuous vibration is generally more perceptible than the impact vibration. Although the vibration level on office floors is usually quite high when compared to residential spaces, the vibrations which occur on office floors are allowable. Since the measurements for the amplitude which was found in each of the studies were different, the amplitude was converted into a displacement amplitude (a peak value) for comparison.

It is quite difficult to make a simple comparison of the results obtained from each of the studies, due to the fact that there were differences in the continuing time for the vibrations, the testing method, the evaluation of the results and the purpose for which the floors are used. However, through the investigation of the correlation between each study's result, from the viewpoint of the evaluation of habitability regarding vibrations, the AIJ grade for the evaluation has been established. Fig. 4, in which the data regarding various floor vibrations recorded in Japan is plotted, shows an example of the habitability evaluation for floor vibrations made according to the AIJ guidelines. It is recognized that most of the data is plotted in the area under the V-5 curve and that each floor functions as a sound floor.

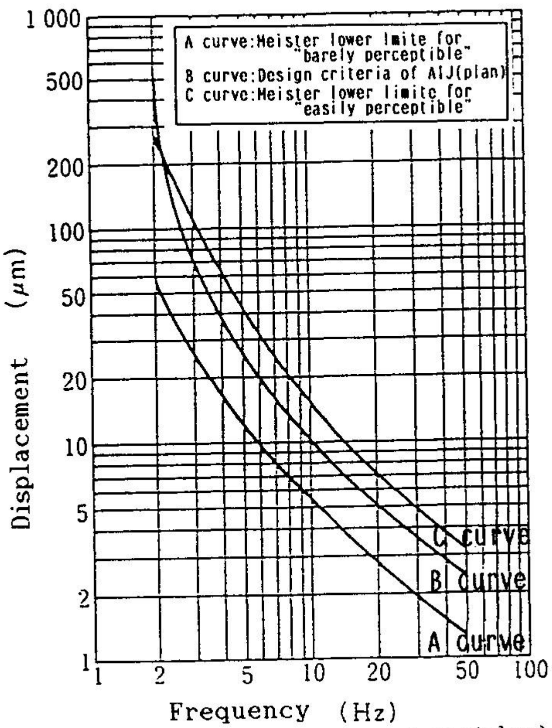

Fig.3 Design criteria of AIJ(plan)

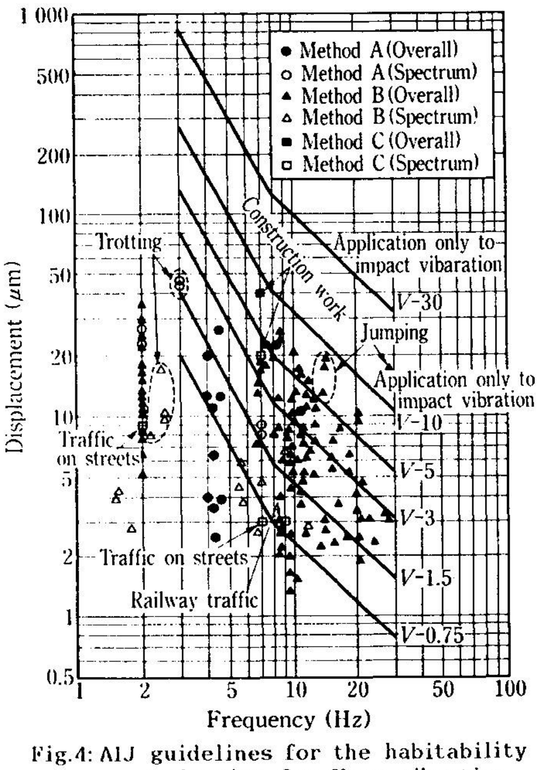

Fig.4: AIJ guidelines for the habitability evaluation for floor vibration.

4. METHOD FOR THE RESPONSE EVALUATION

The evaluation of the efficiency for the floor vibration is carried out through the use of the response wave to the excitation source which is sufficient to expose the objective floors. The AIJ guidelines prescribe the following three response evaluation methods.

a) A method based on the assumed excitation source (Response evaluation method A)
The analytical result of the response caused by the excitation external force which is assumed depending upon how the floor is to be used at the planning stage , is utilized in this method.

b) A method based on a vibration test (Response evaluation method B)
The floor response wave recorded in a vibration test is used. The vibration test in this case is carried out under consideration of an excitation source (the impact induced by one person walking and the impact induced by the footsteps of two people) which was assumed depending on the manner in which the floor is to be used when the building frame is completed.

c) A method based on an actual excitation source (Response evaluation method c)
The floor response wave recorded by the motion of an actual vibration source is used. This actual vibration source is considered to exert a vibrational influence on the floors from both the outside and the inside of the building, after the utilization of the floor begins.

Fig.-5 shows one of the examples of the load imposed upon the floor by a person walking, as well as the walker's response wave. The walk-induced external force acts on the floor repeatedly with the impact generated by the walker's foot steps, and the variation of the load caused by the movement of the walker's body. The period required for a single step is 0.5 sec. In the Fourier spectrum, it is seen that the walking step with 2HZ and the natural frequency of the floor are predominant. However, in order to evaluate the floor efficiency regarding the habitability through the use of a time history wave, the natural frequency of the floor and amplitude A are obtained and are plotted in the evaluation standard. Amplitude A can be obtained from the maximum amplitude 2A in the area where the natural frequency becomes predominant. This method seems to conform to the actual state for the floor vibration.

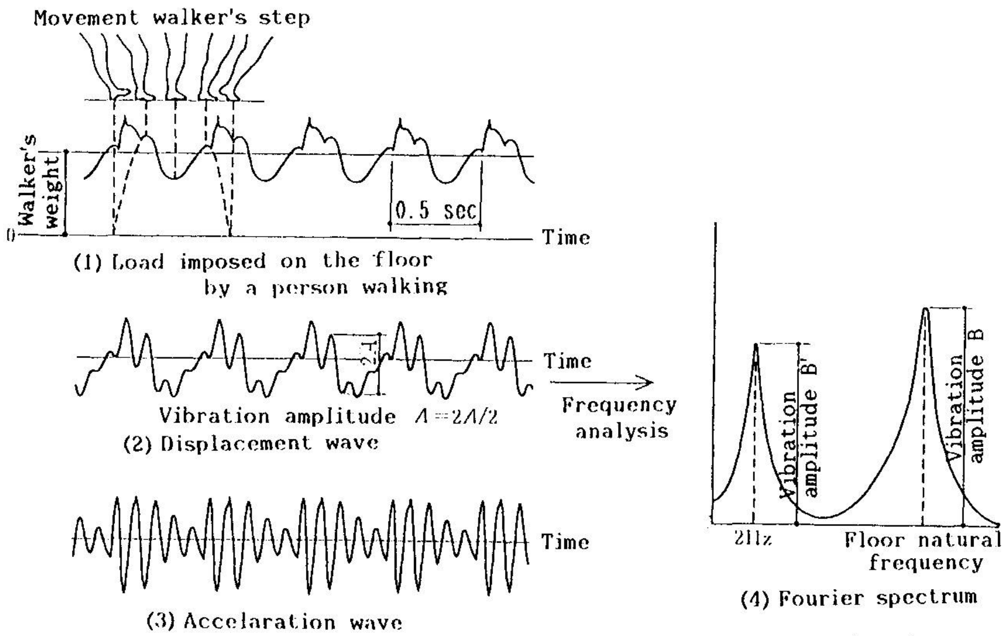

Fig.5: An example of the load imposed on the floor by a person walking and the walker response wave

5. SUBJECT FOR THE FUTURE

The guidelines of the AIJ aim to both clarify the vibration efficiency regarding habitability in a structural plan, and to offer data which is required for the judgment of the evaluation of the vibrational environment in existing facilities. At the same time, the guidelines intend to collect data which has been obtained through actual measurements of vibrations acting on floors of existing buildings in a standardized manner. When data can be accumulated through the application of the standardized method, which will be indicated in the guidelines, in the future, the confirmation and the review of the guidelines may be carried out. Consequently, the maintenance of the environment for the habitability against vibrations in buildings will be assured of attaining a high level.

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Note 1) GSA: General Services Administration Washington U.S.A.
Note 2) CSA: Canadian Standard Association