Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 69 (1993)

Rubrik: Theme 1: Performance requirements

THEME 1

PERFORMANCE REQUIREMENTS

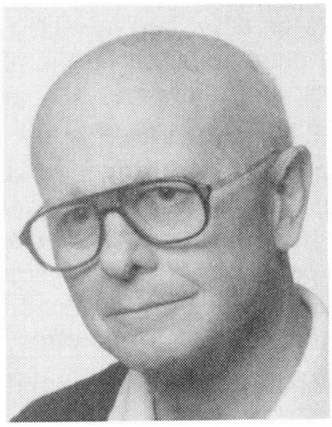
Leere Seite
Blank page
Page vide

Backgrounds to Serviceability Requirements

Considérations sur les conditions d'aptitude au service

Zuverlässigkeitsbedingungen für die Gebrauchstauglichkeit

Milik TICHÝ
Prof. of Civil Eng.
Prague
Czech Rep.



Milik Tichý, born 1929, obtained his engineering degree at the Czech Technical University in Prague. He has been involved in research problems of structural concrete, loads, and structural reliability in general. He has given courses on Reliability and Quality Assurance in Construction. He is currently a Reliability and Forensic Engineering expert.

SUMMARY
Physical, probability-based, and design-reliability requirements used in serviceability limit states design are discussed with emphasis on the constraint problem. Dependencies of constraints upon calculation models, importance of the building, and upon time are considered.

RESUME
On analyse les conditions de sécurité physiques et probabilistes et les conditions de dimensionnement utilisées dans les calculs aux états limites de l'aptitude au service avec une attention particulière pour les limites des variables considérées. La dépendance des limites en vue des modèles de calcul, de l'importance des ouvrages et du temps est considérée.

ZUSAMMENFASSUNG
Es werden die physikalischen, probabilistischen und Zuverlässigkeitsbedingungen betrachtet, die in der Bemessung nach dem Grenzzustand der Gebrauchstauglichkeit insbesondere unter Zwangsbeanspruchung benützt werden. Die Abhängigkeit der Bemessungswerte von dem Berechnungsmodell, der Wichtigkeit des Gebäudes und der Zeit werden untersucht.

1. RELIABILITY REQUIREMENTS AND CRITERIA

To understand correctly the backgrounds of the serviceability requirements applied in the design of buildings, it is necessary to get acquainted with the principal concepts of the "grammar" of the reliability-based design. Let us introduce here some basic concepts (for more detailed information see [3]).

Assume a *fully defined constructed facility* (for example, a building) that does not contain any uncertainties and indefiniteness; the properties of its three basic components, that is the *structure, load,* and *environment* are perfectly known. When this *reliability system* is to be assessed, the *relations* between the components have to be described in such a way we can decide whether the system is reliable or not. These relations must be based on the physical description of phenomena entering the particular system components, and, therefore, they are called *physical reliability requirements.* - In the following, the abbreviation *RelReq* is used for "physical requirement."

In general, scalar variables and vectors of distinctive kind form the physical RelReqs. The nature of these variables and vectors is denoted as *design criterion.* In the main, the serviceability design criteria are expressed in terms of *strain load-effects* (for example, the mid-span deflection, frame sway) and in terms of *vibration parameters* (for example, eigenfrequency, vibration velocity, acceleration). Criteria may also be quantities that are not load-effects; we may state RelReqs in terms of the depth of a beam cross-section referred to the effective span, etc.

In the serviceability design, scalar RelReqs are mainly used; two types of scalar RelReqs are encountered:

- open RelReq:

$$\forall t \in T_{ref}: \quad A(a_1, a_2, \dots, a_n) \leq C \tag{1}$$

- range RelReq:

$$\forall t \in T_{ref}: \quad C_i \leq A(a_1, a_2, \dots, a_n) \leq C_s \tag{2}$$

where $A(.)$ = a quantity described by a physically defined function (or, *calculation model*), a_1 through a_n = elementary variables (called also *basic*), C = constraint, C_i, C_s = lower and upper bound of constraint, respectively; t = point in time, T_{ref} = reference period during which the particular RelReq must be satisfied (for example, life of the building, T_0).

RelReq (1) is typical for the majority of serviceability problems; RelReq (2) is used in design exercises where dynamic behavior of the structure is dealt with. We will not discuss the latter RelReq any more; all conclusions related to RelReqs of type (1) are valid also for type (2).

When, owing to *uncertainties and indefiniteness,* properties of the system investigated are not exactly known, this fact must be taken into account. Therefore, physical RelReqs must be either adjusted by parameters covering uncertainties and indefiniteness, or supplemented by further requirements. When the adjustments are based on experience, or also on theoretical considerations but without regard to the randomness of phenomena, the respective requirements are *deterministic.* If, however, the system uncertainties are treated as random, they can be expressed in terms of the *probability* *of occurrence of adverse realizations of the respective phenomena.* Then, *probability-based reliability requirements* can be formulated:

$$\forall t \in T_{ref}:\ \Pr(C - A \leq 0) \leq P_{ft} \tag{3}$$

where P_{ft} = the target value of the failure probability, P_f.

By synthesis of physical and probability-based RelReqs the *design requirements* are obtained. These are contained in the design codes, or, for particular cases, can be individually specified.

The general form of design requirements is

$$\forall t \in T_{ref}:\ A[F_d^{SLS}(t),\ R_d^{SLS}(t),\ G_{int};\ \mathrm{BC};\ \mathrm{EP};\ t] \leq C \tag{4}$$

where F_d^{SLS} = design values of load considered in the SLS design, R_d^{SLS} = material characteristics (elastic moduli, strengths, creep factors, and others), G_{int} = intended dimensions of the structure; BC stands for boundary conditions, EP for environmental parameters (for example, temperature, humidity). Since RelReq (4) is currently used, we will not analyze it here in detail.

2. RELIABILITY AND DESIGN PARAMETERS

The quantities specifying the intended reliability level are called *reliability parameters.* RelReq (1) through (4) show that two principal reliability parameters must be considered:

- the *reference period,* T_{ref}, which is usually taken as the value of the life expectancy of the building, T_0,
- the *target failure probability,* P_{ft}, in its yearly form, $\dot{P}_{ft}$, or comprehensive form, $\bar{P}_{ft}$; the latter must be referred to a reference period $T_{ref} > 1$ year, for example, $T_{ref} = T_0$.

The values of T_0 and P_{ft} cannot be derived from the properties of the building or of the bearing structure. They have to be determined by *decisions* based on opinions and needs of individuals, groups, and social entities, supported by economic analyses, and, particularly, backed by experience gained with similar facilities. Decisions on T_0 and P_{ft} are not simple since many aspects have to be pondered. The principal aspect is, without any doubt, the *importance of the facility for individuals and the society.* Unfortunately, as far as the serviceability limits states, SLSs, are concerned, we are not yet clear on what values of P_{ft} should be considered. No special studies have been carried out, though for the ultimate limit states, ULSs, well formulated conceptual approaches already exist. Aspects governing P_{ft} for SLSs differ substantially from those related to ULSs. The main difference consists in the fact that according to general opinion, ULSs shall *never* be reached, while the attainment of SLSs can be *sometimes* tolerated.

3. LOADS AND MATERIAL PROPERTIES

Not too much attention has been paid to the design values of load that should be considered in the SLS design. As a rule, characteristic values, that is, 0.95-fractiles of the respective probability distributions, are used. In general, this is not correct, because at the SLS level the "average" loading conditions prevail. Thus the load values introduced should be defined by the mean, mode, or median of the *physical realizations* of load.

Analogous considerations can be made as far as material characteristics are concerned. It is amazing that great care has been paid to the definition of calculation models (for example, for bending stiffness and creep) but the problem of probability-based values to be included into these models is neglected.

4. CONSTRAINTS

In the majority of cases, constraints are specified by *fixed, decision-based values.* Constraint values that have been established in existing design codes have been derived in various ways. In the beginnings of codified design, most of C 's were based on traditions; nobody could give any scientific justification for the respective magnitudes. Now, the situation has been slowly changing, since statistical and probability concepts, the system of reliabilistic thinking, and, last but not least, practical needs have brought new ideas into the constraint issue. As for constraints, modern codes become open-minded, and allow or even encourage the designer to adjust values given in the respective code clauses whenever it is reasonable. Thus, occasions when designers themselves are compelled to specify a constraint value, are getting more and more recurrent. It then happens that the designer, having reached at the conclusion that some constraint is to be verified in the particular situation, founds the available design code unsatisfactory, as for the information given. Then, the designer has to answer *two questions:*

- What shall be the physical meaning of the constraint, C, or in other words, what criterion shall govern the RelReq?
- What shall be the magnitude of C?

In general case, several design RelReqs (4), formulated for *various deformation criteria,* have to be checked. Only in very simple cases, such as floor beams, floor slabs, etc., a single deflection check is sufficient. During the evaluation we must not forget that deformations should be verified also for *several stages of the construction process,* not only for the stage of current use. Further, we must keep in mind the *time-dependencies* involved: first, those related to loads, then those related to material (including soil), and finally also the time-dependence of constraints themselves. The latter is usually underestimated; it will be discussed below.

It is now acknowledged that constraints are, in general, random variables, or more exactly, that they can be established by statistical analysis of aspects which determine their values.

■ **Example 1.** A lecture hall is regularly visited by a group of N individuals. Owing to time-dependent properties of the bearing structure the deformation of the floor grows with time. Let us take the mid-span deflection, f, as deformation criterion. At a certain value of f one of the regular visitors becomes disturbed and begins to be suspicious about the *safety* (not serviceability!) of the structure. Obviously, the respective value of f is the *personal constraint*, f_{lim}, of the visitor. When the deformation continues to grow, the number of alarmed visitors, n, increases. At each lecture, additional Δn visitors will observe the dangerous deflection (let us assume that sensitive visitors' worries are not transferrable). The alarm process is discrete, though the growth of the deformation is continuous; however, the periods when lectures are given are intermittent. The probability that a randomly selected visitor will get annoyed by $f \leq f_{lim}$ is given by

$$P = \frac{n}{N} \qquad \text{(a)}$$

and the probability that a randomly selected visitor will get annoyed just when f_{lim} has been achieved is

$$p = \frac{\Delta n}{N} \qquad \text{(b)}$$

Obviously, each individual has a *personal threshold,* whose exceedance arouses discomfort. As psychological and emotional properties of humans are random, the personal limit deflection, f_{lim}, is also a random variable. Considering a very large population of individuals, Equations (a) and (b) can be written as

$$P = \Phi(f_{lim})$$

$$p = \varphi(f_{lim})$$

where, T_{ref} $\bar{P}_f$ $\Phi(.)$ and $\varphi(.)$ = cumulative distribution function and probability density function, respectively, of the random variable f_{lim}. Consequently, were the probability distribution of f_{lim} known, the value of admissible deflection, f_{adm}, could be find for an intended probability P_{lim} from

$$\Pr(f_{lim} \leq f_{adm}) = P_{lim}$$ ■

Unfortunately, experimental information on random behavior of constraints is still very scarce, or nil. This fact compels us to establish values of constraints, often called "admissible deflections," "admissible crack width," etc., on empirical considerations. Methods, based on the *fuzzy set theory*, are now available that can raise the empiricism to theoretical level [2].

When no guidance on constraints is found in codes and other documents, the designer should ask qualified persons, acquainted with the problem area, for advice. *For example,* we can get

- from *civil and structural engineers:* admissible displacements and deformations with regard to bearing and non-bearing structures that are adjacent to the building designed;
- from *mechanical engineers:* admissible displacements of elevators, piping, etc., that will not impair safe function of the equipment;
- from *agricultural engineers:* admissible deflections and vibrations that do not fright animals stalled. Etc.

However, data supplied shall be always checked for consistency, and the background of such data should be known. It happens that we are offered data on admissible deformations and displacements that are either exaggerated, or, on the contrary, understated.

As for vibration parameters, not only engineers are the source of decisions on constraints. In case of buildings, admissible vibration parameters are, as a rule, specified by *health regulations.* Many designers are unhappy with the prevailing rules, which are often based on concepts different from those built-up in the structural reliability area. Mutual understanding of engineers and hygienists is needed.

Cracks are a phenomenon encountered in all materials. However, only concrete and masonry structures and also structures made of other brittle materials are subjected to serviceability RelReqs based on the occurrence and width of cracks. Considering the crack width as a constraint, we have to take into account that cracks in building structures, for example, may

- be a starting factor in *material corrosion;*
- deteriorate the *sound-proofing* and also *odor-proofing* of partition walls;
- cause *annoyance of the users* of the building.
- impair the *fireproofing* of the building.

Similarly as in the case of deflections, a sensitivity threshold can be found both for structures and for people involved. This threshold can be expressed simply in terms of a limit crack width, w_{lim}, which again is a random variable. Its admissible value, w_{adm}, can be found in the same manner as that of the admissible deflection, f_{lim}; see Example 1.

We should mention here that the *crack width* need not always be the actual governing quantity. Individuals never evaluate the crack width in terms of a physical distance of the opposite faces of a cracked body; their attitude to a cracked structure depends on many factors: *length, shape,* and *density of cracks.* It happens, that a crack of considerable width, say 3 mm, escapes any attention of users and even inspection engineers. When aesthetic reasons affect the admissible crack width, this is only a simplified criterion. A more suitable criterion would be, say, the area of visible cracks per 1 m^2.

We always must keep in mind that cracks are an *unavoidable phenomenon.* Therefore, when a 100%-proof protection against sound, odors, and fire is to be assured, sealing of cracks before the building is put in use must be provided. Then, in the design, delayed movements in cracks due to temperature changes, shrinkage, and other time-dependent effects should be verified taking into account properties of the sealant applied. The same refers to joints that can open because of deformations (for example, joints between partition walls and supporting floors). To facilitate repairs, it is a good practice to assure access to all places sealed.

5. DEPENDENCIES

Various physical and statistical dependencies can be identified in the calculation models for A; for example, the dependence between the elastic modulus and creep factor of concrete. These dependencies are sufficiently known and do not induce any difficulties; they are, in the main, neglected.

However, there is a substantial dependence between the calculation model, $A(.)$, and the constraint, C, though it is not acknowledged in codes. When mandatory values of C are specified by a code, they must be considered valid only for the calculation model given. It happens that a change in calculation model can substantially affect the results of design. Members that were acceptable according to old calculation models, become suddenly unreliable when verified by the new model. In general, this holds also for ULS calculation models, which, fortunately, are not so sensitive as the SLS models.

The above "meta-dependence" between A and C can be source of legal problems whenever neither calculation model nor constraint are specified. Contract documents should always be clear on acceptable deflections, which should be preferably specified on the performance basis, not on calculation model basis.

6. IMPORTANCE

When considering the background of RelReqs, the importance problem should not be ignored. In fact, importance of buildings is not directly expressed in the codified SLS design. No *importance factors* for SLSs are used because the importance and purpose of the building or its part is *embedded in values of constraints specified.* Higher importance is expressed not only by more conservative constraint values than the usual ones but also in the *number of RelReqs assessed.* Performance demands on floors under gymnasiums, dancing halls, assembly halls, and others are much more rigorous than on floors under and over apartments. The difference in importance can be easily considered in the probability-based design.

■ **Example 2.** Consider a hypothetical building with 1000 rooms. The building is used by 1000 persons, each person being allocated to one room at random. Assume that one of the persons is sensitive to any crack in the ceiling, while no cracks are ever registered by any of the remaining persons. Obviously, an event $E_1 \equiv$ Ev(**sensitive person in a room**) is considered. Assume further that also $E_2 \equiv$ Ev(**occurrence of cracks in a particular ceiling**) is random. The floor slabs have been designed exactly so that the target probability of crack occurrence in a slab during the life of the building is $\bar{P}_{rt}$ = 1.0E-3. Obviously,

$$\bar{P}_{rt} = \mathrm{Prob}(E_2)$$

Now, the probability that the crack-sensitive person will become a user of a particular room is

$$\bar{P}_{sp} \equiv \mathrm{Prob}(E_1) = \frac{1}{1000} \equiv 1.0\mathrm{E}{-3}$$

Since in this P_f case E_1 and E_2 are independent and discrete, the *serviceability failure probability* is

$$\bar{P}_{f1} \equiv \mathrm{Prob}(E_1) \cdot \mathrm{Prob}(E_2) \equiv \bar{P}_{sp} \cdot \bar{P}_{rt} = 1.0\mathrm{E}{-6}$$

Consider now the entrance to the building. The reinforced concrete frame is visible and it has been designed for the same cracking probability, 1.0E-3. All 1000 users of the building, including the sensitive one, pass daily through this entrance. If a crack in the frame occurs, it is surely noticed by the sensitive person, and so P_{sp} = 1. Thus, the failure probability is

$$\bar{P}_{f2} = 1.0\mathrm{E}{-3} \times 1 \equiv 1.0\mathrm{E}{-3}$$

The discomfort of the public is substantially different in both cases; in the rooms only a single user will feel uneasy because of the crack appearance, whereas almost all users will become aroused by the crack in the entrance hall (the sensitive person will tell the colleagues about it) with a probability 1.0E-3. Consequently, if for the two facilities the same level of reliability should be achieved, the concrete frame should be designed for $\bar{P}_{rt}$ = 1.0E-6. ■

At buildings used by public the possible discomfort of people is always greater, and so higher levels of reliability have to be used than for buildings used by individuals or small groups.

7. TIME

The time affects the serviceability RelReqs in three ways. First, variables entering the calculation model are time-dependent, each to a certain degree; as a rule, time-dependence of elastic modulus of steel, of structural dimensions, and others is not considered in the calculation models.

Second, some RelReq can govern the design in only the initial periods of the existence of the building and can be entirely ignored later. Therefore, the RelReqs and also the design criteria can differ during the construction and use periods. This is typical for assembled systems where in the erection phase demands on stiffness can vary from operation to operation. Thus, the number and criteria of RelReqs change with time.

Finally, also the constraints can be time-dependent; this fact has not been considered in codes yet. For example, the older the building, the less sensitive the user is to deflections. The deformation of timber frames accumulated in 50 years of service would be unacceptable if it would occur in the first day of service. A client buying an old farm house to spend holidays and vacation there is little sensitive to large deflection of floor beams, considering it unavoidable. The same effects can be observed as far as cracks are concerned. When the deflection and crack width grow slowly and steadily, the owners and users do not become suspicious about safety of the building even when their magnitude is high.

8. CONTEXT

The subjective assessment of existing deflections, crack width, and other serviceability criteria is always a part of the *risk assessment.* The actual risk is evaluated along a large scale of values, starting with simple repeated costs necessary for current maintenance and ending with costs involved with the evacuation of the building.

In the main, the risk assessment is carried out by users (for example, tenants living in a residential building) in the first plane, than also by owners (landlords, farmers), and *in extremis* by reliability experts. The users' assessment is virtually subconscious, but later it becomes more and more specific. The owner's assessment is based on economic thinking, and the reliability experts make benefit of their theoretical knowledge and experience. The nature of the assessment is successively psychologic, economic, and scientific. It is felt that some general *risk units* should be introduced; in the absence of such, monetary units can serve the purpose.

Observe that people are the principal component of the assessment process. Thus, the assessment is exposed to subjective attitudes the complex of which is called the *context* (see Elms in [1]). The results of assessment and the ensuing actions taken depend upon the context substantially. At the same situation, different decisions will be made by users, owners, and experts. The evaluation of deflections, cracks, vibrations, and further serviceability phenomena will be different with men and women, users and owners, old people and young people, etc.

The foregoing paragraphs have shown the variety of problems encountered with serviceability reliability requirements. These problems are manifold; we can maintain that they are more diversified than those associated with ultimate reliability requirements.

REFERENCES

1. Engineering Safety, Ed. by D. Blockley. McGraw-Hill, London, 1992, 475 pp.
2. HOLICKÝ M., Optimization of structural serviceability. *Stavebnícky časopis,* Vol. 39, 1991a, No. 9-10, pp. 473-486.
3. TICHÝ M., *Applied Methods of Structural Reliability.* To be published in 1993 by Kluwer Academic Publishers, Dordrecht, The Netherlands, about 400 pp.

Leere Seite
Blank page
Page vide

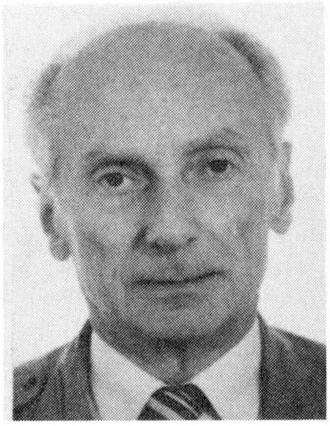
Serviceability Requirements

Exigences d'aptitude au service

Gebrauchstauglichkeitsanforderungen

György DEÁK
Professor
Technical University
Budapest, Hungary

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



György Deák, born 1926, got his cvil engineering, architect and doctor degrees at the Technical University of Budapest. For long time he was involved in education and research of reinforced concrete and prestressed concrete structures, their quality control and serviceability analysis.

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.

SUMMARY

Serviceability requirements imposed on building structures are classified into three basic groups: human comfort, structural and equipment requirements. To specify limit values and to analyze structures two kinds of uncertainties are to be distinguished: vagueness in definition of limit states and randomness of basic variables. Methods of optimization may provide some guidance. Calculation, testing and professional judgement could be used for verification and assessment of structural serviceability.

RESUME

Les exigences d'aptitude au service imposées aux structures des bâtiments sont réparties en trois groupes: confort de l'utilisateur, exigences d'ordre structural et exigences relatives aux équipements. Il faut tenir compte de deux sortes d'incertitudes au cours de la détermination des valeurs limites et du calcul des systèmes porteurs, à savoir l'imprécision des états limites et le caractère aléatoire des variables de base. Les méthodes d'optimisation peuvent fournir à ce sujet une aide appréciable. Le calcul, l'expérimentation et l'appréciation du spécialiste sont d'excellents atouts pour évaluer et vérifier l'aptitude au service d'une structure donnée.

ZUSAMMENFASSUNG

Anforderungen an die Gebrauchstauglichkeit von Hochbautragwerken können in drei Klassen eingeteilt werden: Benützerkomfort, bauliche und einrichtungstechnische Anforderungen. Bei der Festlegung von Grenzwerten und der Berechnung sind zweierlei Unsicherheiten zu berücksichtigen, die Unschärfe der Grenzzustände und der stochastische Charakter der Basisvariablen. Optimierungsmethoden können hier Entscheidungshilfe geben. Berechnung, Experiment und Urteilsfähigkeit sind einsetzbar, um die Gebrauchstauglichkeit eines Tragwerks abzuschätzen und nachzuweisen.

1. INTRODUCTION

Serviceability of building structures and other civil engineering works is a broad concept, whose extent seems to be continuously expanding. This is caused by several trends in design and production of building structures as well as by increasing demands on their performance. Structural serviceability should cover an essential part of the overall structural performance, which, in accordance with the International Standards ISO 2394[1], ISO 6240[2], ISO 6241[3] and ISO 4356 [4], includes two basic groups of mechanical properties

-- safety or load bearing capacity, i.e. resistance to various actions without collapse or total disability of the structures or their elements,

-- serviceability, i.e. ability of structures and their elements to perform adequately in normal use.

Obviously, boundary between these two mechanical properties is not absolutely sharp and entirely unambiguous. Durability and fatigue of structures are examples of phenomena, that are frequently included in both these groups. Generally, it is understood, that violation of appropriate limit states of safety may cause risk of human life and malfunction costs many times exceeding the initial costs, whereas violation of serviceability limit states rarely lead to risk of human life and usually involve lower economic losses than in case of safety.

On the other hand overwhelming majority of structural defects observed nowadays, are classified as serviceability, rather than safety problems. That is why serviceability limits states are becoming more and more important technical as well as economical issue [5,6,7,8].

While safety problems usually involve strength, serviceability problems involve primarily deformations and displacements of different origin. It is to be noted here, that there are generally two independent sources of dimensional changes, that should be, in some cases, taken into account simultaneously when analyzing structural serviceability: deformations due to various actions including loads, and deviations due to various production procedures including setting out, manufacturing and erection. It follows from another contribution at that colloquium [9,10], that common procedures for dealing with structural serviceability are insufficient and need to be improved. However similar statement follows from other serious drawbacks of the current methods [5,7].

The underlying aim of this document is to unify basic classification of serviceability requirements, formulation of adequate criteria and general procedures for design and assessment of building structures with respect to the serviceability limit states. It is believed that some general guidance towards uniformity in specification and required probability of compliance with imposed requirements will be welcome, particularly, as the economy of modern buildings are increasingly controlled by their serviceability.

2 SERVICEABILITY REQUIREMENTS

Serviceability requirements should guarantee adequate performance of the building in normal use [1]. In general, serviceability requirements commonly imposed on buildings and civil engineering works, could be classified into the following three basic groups, which correspond to the functional requirements specified in the International Standard ACE 6241/3/, in several national standards as well as in working documents of developing international codes:

(1) human comfort, which may be further divided in two subgroups:

-- appearance requirements (to limit annoying visual effects due to deformations and cracks of structural components),
-- physiological requirements (to limit discomfort due to vibration, penetration of air, dust and sound);

(2) structural requirements (to limit local damage including stress, strain, excessive cracks and to guarantee, smooth assembly, watertightness, drainage and proper functioning of attached elements, coverings, doors and windows);

(3) equipment requirements (to guarantee proper functioning of all kinds of equipment, including machinery, pipes, cables, ducts and their supports).

These basic groups comprise typical serviceability requirements, which are most frequently imposed on newly designed structures. Explanatory examples in brackets may help, but obviously are far from being exhaustive. They may serve as an aide-memoir to identify all appropriate functions of structures and to specify adequate serviceability requirements.

Indicated basic groups of serviceability requirements are obviously overlapping and/or criteria derived from them may be mutually dependent or interactive. This may result in complex general criteria dependent on span of the components or other relevant characteristic. In particular cases, however, often only one type of serviceability requirement is decisive in design and assessment of structural serviceability.

3. SERVICEABILITY CONDITIONS

It is a common rule that serviceability requirements lead to criteria for adequate deformations, displacement or other mechanical indicators, which are called serviceability parameters. To identify relevant serviceability requirements and their quantitative specification in terms of suitable serviceability parameters is the most important and difficult task of design and assessment of structural serviceability.

The serviceability parameters u_i are suitable mechanical variables (as for example deflection at midspan, slope at a given point, acceleration, crack width), which should characterize ability of a structure to be used for the purpose for which it is intended. Usually only one serviceability parameter u, or two parameters u_i are considered for a structure at a time [10].

Serviceability requirements should be then expressed in terms of the chosen parameters u_i as serviceability conditions, usually in the form of simple inequalities between the actual (calculated) structural values $z_i(t)$ of the parameters u_i and their limit values (constrains) l_i, t being time. Most often, the serviceability conditions state, that the actual structural value $z_i(t)$ of the serviceability parameter u_i should not exceed, or may exceed only within a limited time period, specified limit values l_i.

The most frequently applied serviceability criterion, concerning just one parameter u, has the following simple form

$$z_i(t) \leq l_i \;. \tag{1}$$

In some cases, however, more complicated criteria, including both upper and lower limit values [10], or concerning a set of parameters u_i and corresponding actual values $z_i(t)$ and the limit values l_i, may be applied. The limit values l_i are dependent on the building occupancy, considered time period and on the reversibility of the caused unserviceability. Generally, however, they are not dependent on the material used for the load bearing structure, and may be usually considered as time independent quantities.

4. UNCERTAINTIES

There are two kinds of uncertainties to be considered when analyzing serviceability limit states:

-- vagueness in the definition of serviceability limit states, as in most cases unserviceability develop gradually with increasing value of appropriate parameters,

-- randomness of loads, mechanical and geometric characteristics, sensitivity of occupants and attached structural components and equipment.

While randomness of basic and resulting variables can be handled mathematically through the well established theory of probability, less familiar imprecision and vagueness in definition of limit values l_i may be handled by methods of newly developing theory of fuzzy sets [11,12,13,14].

Thus to analyze serviceability limit states a probabilistic approach should be used similarly as in the case of ultimate limit states. In the latter case the annual probability of failure is of the order 10^{-5} to 10^{-6}, in the former case the annual probability of exceeding serviceability conditions (unserviceability) is of the order 10^{-3} to 10^{-2} or even greater. However, if the consequences of unserviceability are serious (hospitals, power plants, etc.), then unserviceability should be allowed only with approximately the same probability as in the case of ultimate limit states.

Unless methods of probabilistic analysis and structural optimization [16,17] will provide more accurate data, it is recommended to determine design values of the actual structural values of serviceability parameter under the following assumptions

-- actions are considered by their characteristic value (for irreversible consequences as damage of attached nonstructural components), or by frequent value (for reversible consequences as visual disturbance); upper values are taken for unfavourable actions, lower values for favourable actions;

-- dimensions are considered by their nominal values, given in design documentation;

-- materials characteristics are considered by their unfavourable characteristic values (5% fractiles);

-- prestressing force is considered similarly as mechanical properties by 5% fractile.

The limit values of serviceability parameters should be considered by their fractiles or expected total unserviceabilities [9,14] corresponding uttermost to *20%*, in accordance with the significance of possible consequences. In some cases methods of optimization may provide more accurate specification [17,18].

5. STATE OF STRUCTURE

The state of a structure exposed to various physical and chemical causes, including load, is described by time dependent random variables (functions) $z_i(t)$ representing actual induced deviations and structural response to various actions expressed in terms of serviceability parameters u_i. As mentioned above the state of a structure is affected by both, deformations and deviations. Generally the random function $z_i(t)$ should consequently include effects of time dependent deformations of structural components due to physical and chemical causes including load, as well as effects of deviations induced by setting out, manufacturing and erection [10].

An actual structural value $z(t)$ of the serviceability parameter u (deflection, amplitude), may be a monotonic (irreversible) or fluctuating (partially reversible) function of time. At any time t, the variable $z(t)$ is a random quantity, which might have a considerable scatter [15,16]. Behaviour of the random function or $z(t)$ is described by a probability density function $\varphi_z(u|t)$ characterized by the mean $\mu_z(t)$, standard deviation $\sigma_z(t)$, skewness $a_z(t)$ and possibly by other statistical characteristics. Positive skewness $a_z(t)$ is likely to be expected for such parameters as deflection and amplitude [3].

To determine reliable statistical characteristics of the variable $z(t)$, appropriate physical and chemical causes including loads, must be considered. Load combinations should correspond to the nature of relevant requirements and specified serviceability parameters. In many cases only approximate values of other various physical and chemical causes are available.

6. LIMIT VALUES

As already mentioned, relevant requirements are usually stated very vaguely, imprecisely, often only verbally and, consequently may be very subjective [9,14,18]. To specify limit values l for serviceability parameters u, the following attributes should be therefore stated:

- considered serviceability requirement,
- structure or structural element to be verified,
- serviceability parameter and its limit value,
- corresponding probabilistic measures (probability or unserviceability),
- design situations to be considered,
- the load combinations to be taken into account,
- recommended simplified rules (e.g. limiting span/depth ratio),
- possible structural solutions including detailing to reduce risk of unserviceability.

This list of attributes seems to be useful to prepare standard specifications and recommendations for verification of structural serviceability and could be included in operational standard documentations, in order to enable alternative specifications. In view of economic aspects, client, contractor or architect may have their own demands different from code recommendations. In such cases mutually agreed serviceability requirements should be specified in a special contract.

7. ASSESSMENT AND VERIFICATION

Assessment and verification of each serviceability requirements may be generally done by means of calculation, test or judgment. The choice depends on the stage of building activity (designed, constructed, completed or old structure) and also on the particular serviceability requirement.

A calculation indicates the extent of satisfaction with serviceability requirements by means of theoretical model of behaviour, which should take into account all sources affecting actual value of the serviceability parameter, as for example creep, shrinkage, development of cracks, plastic deformations, local instability, induced deviations if they occur at appropriate design situation. It is however generally preferable to design the structure in such a way as to limit if not exclude all the possible unfavourable phenomena violating adequate performance and derived serviceability requirements.

A test provides a basis for assessing the satisfaction of serviceability requirements of a structure or structural elements. Direct measurements or other means of determination of the actual value of considered serviceability parameter under either real conditions of use, or conditions appropriately correlated to use, are then employed. A professional judgment or appraisal can permit the extend of satisfaction of serviceability requirements to be assessed on the basis of comparison with well established solutions.

In all cases appropriate reliability over specified time period need to be considered. Accepted level of probability or unserviceability should be related to expected consequences. In some cases structural optimization methods, based on minimum life cycle cost, may provide some guidance [14,16,17].

8. CONCLUSIONS

(1) Serviceability requirements could be classified into the following groups:
- human comfort,
- structural requirements,
- equipment requirements.

(2) There are two kinds of uncertainties to be considered when analyzing serviceability limit states:
- vagueness in the definition of serviceability limit states,
- randomness of loads, mechanical and geometric characteristics, sensitivity of occupants and attached structural components and equipment.

(3) Standard recommendations for limit values should include relevant attributes in order to enable an alternative specification,

(4) Verifications of structural serviceability may be done by calculation, testing or professional judgment.

REFERENCES

1. ISO 2394-1986 General principles on reliability for structures.
2. ISO 6240-1984 Performance standards in building - Contents and presentation.(in course of revision).
3. ISO 6241- 1984 Performance standards in building - Principles for their preparation and factors to be considered.
4. ISO 4356-1977 Bases for the design of structures - Deformations of buildings at the serviceability limit states.
5. GALAMBOS T.V. and ELLINGWOOD B., Serviceability Limit States Deflection. Journal of Structural Engineering. ASCE, 112(1), 1986, pp. 67-84.
6. Proceedings of the CIB Symposium/Workshop on Serviceability of Buildings. NRC Canada, Ottawa 1988.
7. Ad Hoc Committee on serviceability Research. Structural Serviceability: Critical Appraisal and Research Needs. Journal of Structural Engineering, Vol 1/2 N12, pp. 2646-2664, 1986.
8. LEICESTER R.H., On Developing an Australian Limit States Code. In: International Timber Engineering Conference, Japan, Tokyo 1990.
9. HOLICKÝ M. and ÖSTLUND L., Probabilistic Design Concept. In: CIB International Colloquium on Structural Serviceability of Buildings, Göteborg, Sweden, June 1993.
10. HOLICKÝ M. and HOLICKÁ N., Serviceability and Tolerances. In: CIB International Colloquium on Structural Serviceability of Buildings, Göteborg, Sweden, June 1993.
11. BLOCKLEY D.I., The Natura of Structural Design and Safety. Ellis Horwood Limited, Chichester, 1980.
12. MUNRO J. and BROWN C.B., The Safety of Structures in the Face of Uncertainty and Imprecision. In: Fourth Interational Conference on Application of Statistics and Probability in Soil and Structural Engineering. Italy 1983, pp. 695-711.
13. BROWN C.B. and YAO J.T.P., Fuzzy Sets and Structural Engineering. Journal of Structural Engineering, ASCE, May, 1983, pp. 1211-1225.
14. HOLICKÝ M., Fuzzy Concept of Serviceability Limit States. In: CIB Symposium on Serviceability of Buildings. NRC Canada, Ottawa 1988, pp. 19-31.
15. ACI Committee 435 : Variability of Deflection of Simply Supported Reinforced Concrete Beams. Journal of American Concrete Institute, Vol.69., No.5, pp. 1211-1225, May 1983.
16. HOLICKÝ M., Theoretical analyses of random structural deformations. Acta technica ČSAV, Praha, 1975.
17. BROWN C.B., Optimizing and Satisficing. Structural Safety, March 1990, pp. 155-163.
18. HOLICKÝ M., Optimization of structural Serviceability. Stavebnícky Časopis, 1991/9-10.

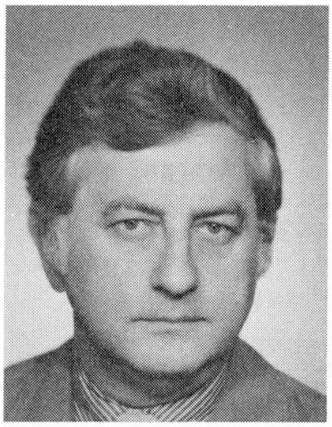
Leere Seite
Blank page
Page vide

Serviceability and Tolerances

Aptitude au service et tolérances

Gebrauchstauglichkeit und Toleranzen

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



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

Năda HOLICKÁ
Civil Engineer
Czech Techn. Univ.
Prague, Czech Rep.

Năda Holická, born 1943, got her civil engineering and doctor degree at the Czech Technical University in Prague, her master's degree at the University of Waterloo, Canada. Năda Holická is now active in teaching and research of concrete structures.

SUMMARY

Two independent sources of dimensional changes should be taken into account when analyzing structural serviceability: deformations due to various actions including loads as well as deviations due to production procedures. General principles for simultaneous consideration of both kinds of dimensional changes are simplified for most frequently applied serviceability requirements.

RESUME

Deux sources de déviations dimensionnelles doivent être considérées dans l'analyse de l'aptitude au service des constructions: déformations dues aux actions sur les structures et déviations causées par des processus de construction. Les principes généraux de calcul tenant compte des deux types de déviations dimensionnelles sont simplifiés pour les cas les plus fréquents du calcul de l'aptitude au service.

ZUSAMMENFASSUNG

Bei der Untersuchung der Gebrauchstauglichkeit von Bauwerken sollten zwei Ursachen für geometrische Abweichungen unterschieden werden, nämlich Verformungen unter verschiedenen Einwirkungen und Herstellungstoleranzen. Für die häufigsten Gebrauchstauglichkeitsanforderungen werden allgemeine Prinzipien der gekoppelten Betrachtung beider Einflüsse sowie ein vereinfachtes Verfahren vorgestellt.

1. INTRODUCTION

Serviceability of building structures is a broad concept, which is affected by all kinds of possible dimensional changes of structural components and elements from their nominal or target configurations. Generally, there are two different sources of dimensional changes:

- structural deformations due to actions, including physical and chemical causes and all kinds of loads - called shortly deformations,
- induced deviations due to production procedures including setting out, manufacturing and erection - called shortly deviations.

Deformations can be affected by some deviations (for example by cross section height and supporting conditions). As a rule, however, this dependence is insignificant and consequently deformations and deviations may be considered as mutually independent sources of dimensional changes.

Compliance with serviceability requirements is generally dependent on both kinds of dimensional changes, on deformations as well as on deviations. Consequently, both kinds of dimensional changes should be considered simultaneously to verify appropriate design criteria whenever mutual interaction of deformations and deviations occurs. The following explanatory examples of serviceability requirements represent typical cases, where such an interaction may arise:

- visual requirements on sag of horizontal components,
- operational requirements on flatness of floors,
- tightness of bed and ceiling joints of partition walls,
- watertightness of cladding joints.

This short list may help to identify other important cases of structural serviceability, where combination of deformations and deviations play a significance role, but obviously is far from being exhaustive.

Current methods of serviceability analyses of building structures neglect entirely the effect of deviations and, therefore, need to be improved. The aim of this contribution is to state relevant fundamental principles and to propose simplified rules for simultaneous consideration of both sources of dimensional changes. It is perhaps the first attempt to include effect of deviations in serviceability analyses and, consequently, developed methods may need further improvement.

2. BASIC CONCEPTS

To verify specified requirements, appropriate serviceability parameters (for example displacement of the midspan point of a horizontal component, width of a joint) are to be identify first. In most cases only one parameter $z(t)$, t being time, could be considered independently of the other serviceability parameters. The basic value of the parameter $z(t)$ is the reference or nominal value z_R, which is specified in the design and to which all kinds of dimensional changes are to be related. It is the time independent value determined for modular and structural requirements without taking into account all kinds of deformations and deviations. In some cases the reference value is zero (sag of horizontal components), in other cases is equal to a certain non zero intended (design) size (width of joints).

As demonstrated above, the actual structural value of the serviceability parameter $z(t)$ may be affected by two separate sources of dimensional changes, deformations and deviations. While displacement of a given point due to structural deformations could be described by time dependent random function $x(t)$, deviations are represented by time independent variable y. Both these quantities are assumed to have normal distribution. The resulting serviceability parameter $z(t)$, is then described by normal random function, which is the sum

$$z(t) = z_R + x(t) + y \, , \tag{1}$$

Generally two limiting values, lower and upper limits l_L and l_U are specified for the parameter $z(t)$ to guarantee compliance with serviceability requirements. Thus, the following serviceability condition is to be satisfied

$$l_L \leq z(t) \leq l_U \, . \tag{2}$$

For the sake of simplicity both limit values l_L and l_U are assumed to be deterministic values, even though, as follows from other contributions at this Colloquium, that there is considerable vagueness in their definition.

As follows from Equation (1), the serviceability parameter $z(t)$ is a random quantity which may be handled by methods of classical theory of probability. Consequently, in order to verify the above condition (2), two probabilities p_L and p_U, which are permitted for overstepping the lower and upper limit l_L and l_U respectively, are to be specified. Equal probabilities for both limits, $p_L = p_U = p$, of the order of 10^{-2} to 10^{-1} are usually proposed for serviceability limit states.

3. DEFORMATIONS AND DEVIATIONS

Deformations are always caused by various time dependent actions, which lead to random structural responses. Using appropriate mechanical models of structural analyses, random function $x(t)$, representing a deformation, could be described by the mean function $\mu_x(t)$ and standard deviation $\sigma_x(t)$. For the purpose of serviceability analyses, the standard deviation may be often approximated by time independent value σ_x. However, actual distribution of structural deformations may have considerable asymmetry (mostly with positive skewness). Then normal distribution is only an approximation and other more suitable probabilistic models (lognormal distribution) are to be applied.

In accordance with principles of accuracy analyses [1], statistical characteristics of deviations are specified by the mean μ_y (systematic deviation from the reference value) and limit deviation δy, which is equal to one half of the tolerance width Δy. The standard deviation σ_y is related to the limit deviation δy or to the tolerance simply as

$$\sigma_y = \frac{\delta y}{k} = \frac{\Delta y}{2k} \, , \tag{3}$$

where the coefficient k depends on the probability p accepted for overstepping the limit deviations; for $p = 0.05$, $k = 1.65$. This concept of tolerance specification is illustrated in Fig.1.

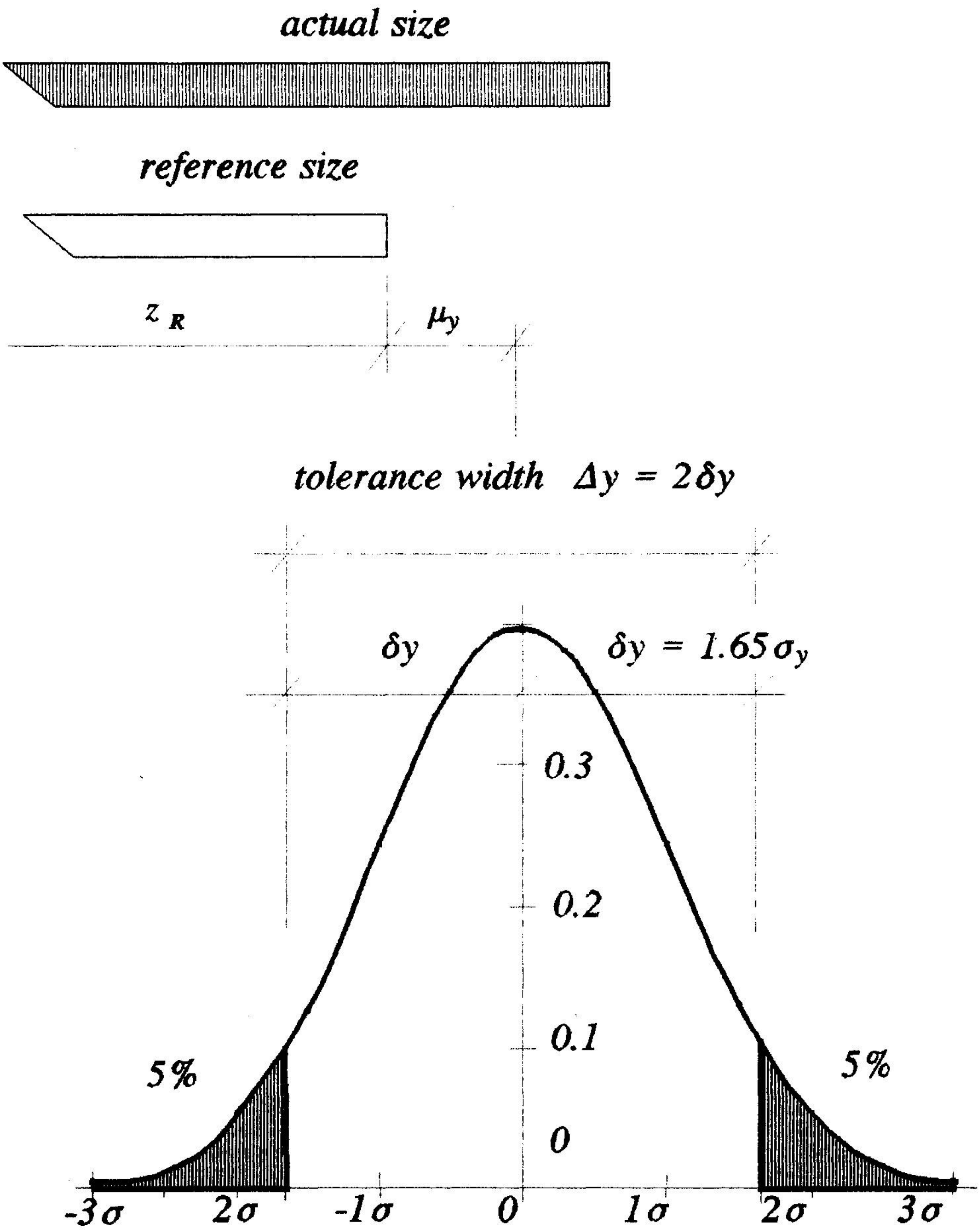


Figure 1 Characteristics of deviations y.

4. SERVICEABILITY CONDITIONS

Statistical characteristics of the serviceability parameter $z(t)$ follow from Equation (1)

$$\mu_z(t) = z_R + \mu_x(t) + \mu_y\,, \qquad \sigma_z^2(t) = \sigma_x^2(t) + \sigma_y^2 \approx \sigma_z^2\,. \tag{4}$$

As indicated by the second relationship of Equation (4), the standard deviation $\sigma_z(t)$ may be usually approximated by time independent value σ_z. General form of the serviceability condition (2) can be then rewritten in terms of the statistical characteristics of the parameter $z(t)$ as

$$z_R + \mu_z(t) - k_L\,\sigma_z \ge l_L\,, \qquad z_R + \mu_z(t) + k_U\,\sigma_z \le l_U\,. \tag{5}$$

where the coefficients k_L and k_U are dependent on the probabilities p_L and p_U; if $p_L = p_U = p = 0.05$, then $k_L = k_U = k = 1.65$.

It follows from Equation (4), that the serviceability conditions given by Equation (5) may be expressed in terms of statistical characteristics of deformations and deviations as

$$\mu_x(t) + \mu_y - k_L\sqrt{\sigma_x^2(t) + \sigma_y^2} \ge l_L - z_R\,, \qquad \mu_x(t) + \mu_y + k_U\sqrt{\sigma_x^2(t) + \sigma_y^2} \le l_U - z_R\,. \tag{6}$$

The above inequalities represent general form of serviceability conditions for one parameter, when both deformations and deviations are taken into account.

In many practical cases, however, the mean μ_y of deviations y (systematic deviation) is zero. Further, using the second row of Equation (4), the standard deviation $\sigma_z(t)$, may be often approximated by a time independent quantity σ_z. Then the mean deformation $\mu_x(t)$ should satisfy the following simplified conditions, which are derived from the previous Equation (6)

$$\mu_x(t) \ge l_L - z_R + k_L\,\sigma_z\,, \qquad \mu_x(t) \le l_U - z_R - k_U\,\sigma_z\,. \tag{7}$$

Note, that if both limits l_L and l_U are specified (for example when width of joints is verified), then the maximum standard deviation σ_z (the optimum case), which could be permitted, can be used if the reference value z_R is related to the mean deformation $\mu_x(t)$ as follows

$$z_R = \frac{l_L + l_U}{2} - \mu_x(t)\,, \tag{8}$$

This relationship is effectively applied in accuracy analyses of assembled structures [1], which are closely linked, if not directly belong, to serviceability limit states (for example case of tightness requirements imposed on internal and external joints). Practical examples, including detail numerical calculations may be found in the book [1], or in other references indicated in [1].

When structural serviceability is analyzed, usually the upper limit $l_U = l$ is considered only. If the reference value z_R is zero (for example in case sag of horizontal components), then the mean deformation should satisfy the following condition

$$\mu_x(t) \leq l_U - k_U \sigma_z \,. \tag{9}$$

which follows from Equation (7). The above Equation (9), might be the most frequently applied criterium for verification of serviceability limit states when effects of deviations are considered.

5. EXAMPLE

The functional requirement on flatness of a floor is specified in as of the permissible deviation z from a straight edge, say $z = 4\ m$ per $2\ m$. Expected accuracy of a specified production technique is $\delta y = 3\ m$ per $2\ m$. In view of Equation (3), its standard deviation is

$$\sigma_y = \frac{3}{1.65} = 1.82\ mm \,. \tag{10}$$

If the span of the bearing horizontal component is $L = 3.6\ m$, then its midspan deflection could be $L/250 = 14.4\ m$ with the standard deviation $2.9\ m$ (assuming coefficient of variation about 20%). The maximum deflection of the span $2\ m$ is about $(2/3.6)^4 \times 12 \approx 1.5\ m$ with the standard deviation $\approx 0.3\ m$. The resulting standard deviation of the parameter z, follows then from Equation (4) as

$$\sigma_z = \sqrt{1.82^2 + 0.3^2} = 1.84\ mm \,. \tag{11}$$

and the maximum permissible mean deflection of a span of $2\ m$ follows from Equation (9) as

$$\mu_x = 4 - 1.65 \times 1.84 = 0.96\ mm \,. \tag{12}$$

Thus the deflection of the span $3.6\ m$ should not exceed $(3.6/2)^4 \times 0.96 = 10.1\ m$, which is about $L/360$.

Mutual interaction of deformations and deviations is obviously dependent on considered serviceability requirement and assumed input data. Nevertheless, the above example clearly indicates, that proposed principles and simplified rules could be virtually applied, and may be, therefore, considered for possible improvement of existing methods commonly applied for serviceability analyses.

6. CONCLUSIONS

(1) In some serviceability limit states both deformations, due to various actions, and deviations, induced by production procedures, must be considered simultaneously.
(2) Existing methods of serviceability analyses neglect entirely the effect of deviations and need to be improved.
(3) Proposed principles and simplified rules provide efficient procedures to include effect of deviations and should be considered when revising present methods of serviceability analyses.

REFERENCES

1. VORLÍČEK M. and HOLICKÝ M., Analysis of dimensional accuracy of building structures, Elsevier, Amsterdam 1989.

Habitability under Horizontal Vibration of Low Rise Buildings

Confort des immeubles bas soumis à des vibrations horizontales

Bewohnbarkeit niedriger Gebäude unter horizontalen Schwingungen

Shinji NAKATA
Eng.
Asahi Chemical Ind. Co. Ltd.
Tokyo, Japan

Yukio TAMURA
Eng.
Tokyo Inst. of Polytechnics
Atsugi, Japan

Tamio OTSUKI
Eng.
Shimizu Corporation
Tokyo, Japan

SUMMARY

Horizontal vibration perception tests are conducted for high-frequency horizontal vibrations expected with mid- and low-rise steel-framed buildings due to road traffic. The test results are evaluated probabilistically.

RESUME

Des essais de perception de la vibration horizontale ont été effectuées, pour des vibrations à haute fréquence dues au trafic routier, sur des bâtiments à ossature métallique de basse et moyenne hauteur. Les résultats des essais sont traités de façon probabiliste.

ZUSAMMENFASSUNG

Es wurden Tests zur Wahrnehmung hochfrequenter horizontaler Schwingungen durchgeführt, wie sie in niedrigen und mittelhohen Stahlskelettbauten infolge von Strassenverkehrserschütterungen zu erwarten sind. Diese Tests werden mit Wahrscheinlichkeitsberechnungen ausgewertet.

1. PURPOSE

Current research efforts and guidelines regarding the response of occupants exposed to horizontal motions and vibration in buildings deal only with low-frequency ranges (0.33-2.0 Hz) [1-5]. This is due to the practical requirements for the design of high-rise buildings. However, mid- and low-rise steel-framed buildings sometimes pose problems related to vibrations caused by road traffic. In this study, horizontal vibration perception tests at high frequencies around 3.0 Hz, which is the normal range for the natural frequencies of mid- and low-rise steel-framed residential buildings, were conducted. Results were evaluated probabilistically to propose guidelines for the evaluation of the habitability of mid- and low-rise steel-framed houses.

2. TEST METHOD

2.1 Test Facilities

The plan of the test room is shown in Fig. 1. The test room was set up on the fifth floor of a seven-story steel-framed experimental tower. Vibration generators were installed on the sixth floor of the tower to create the motion of vibration along one axis as shown in Fig. 1.

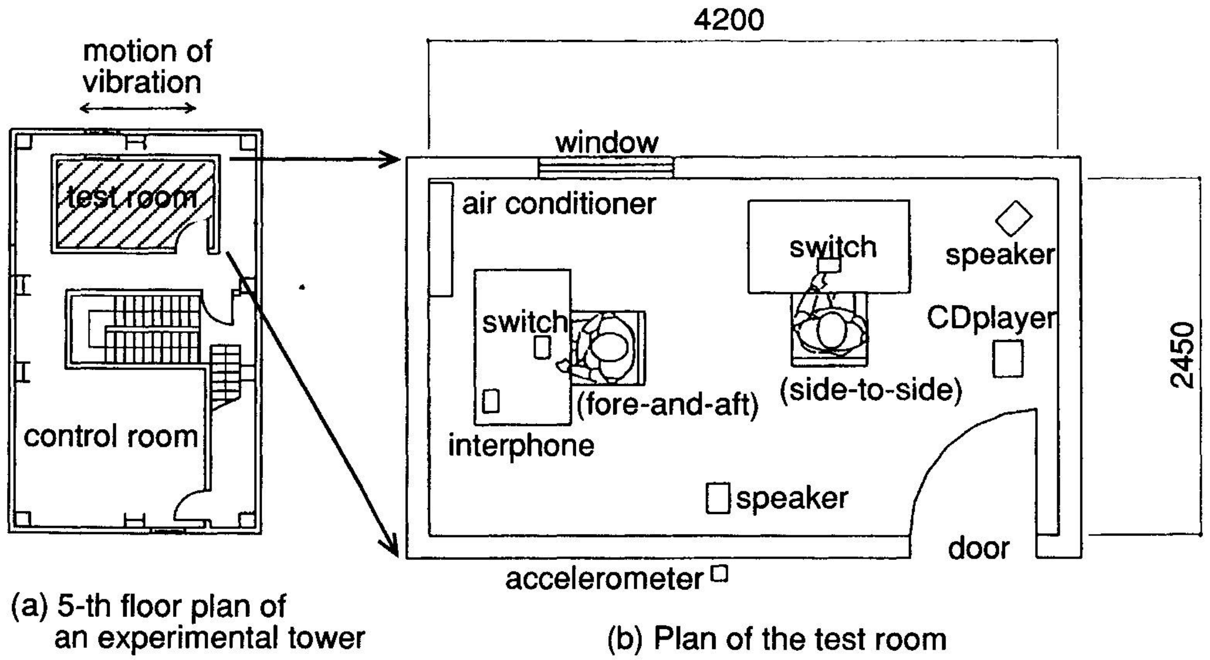
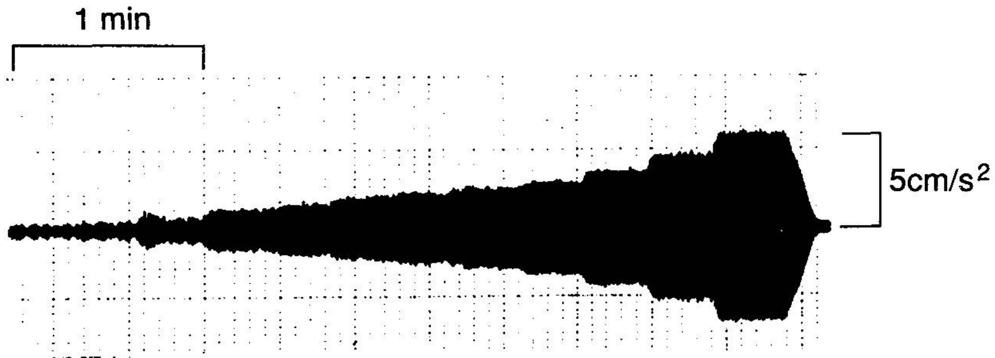


Fig.1 Test facilities

2.2 Test Procedures

The perception tests were conducted on a total of 40 subjects. Vibration was given to a seated subject in both fore-and-aft motion and side-to-side motion. Each subject's perception was detected by telling the subjects to press an "ON" button when he/she started perceiving a vibration. Sinusoidal vibrations of a certain frequency with an increasing amplitude were given. The frequency was set at 1.0, 1.9, 3.0, 4.0, and 6.0 Hz. Fig. 2 shows examples of the response acceleration of the test room at 1.0 and 3.0 Hz. The subjects were told to fill out a questionnaire, which was designed to check if they perceived any vibrations and evaluate how they were perceived, after the test.

(1.0Hz)



(3.0Hz)

Fig.2 Examples of test room acceleration

3. EXPERIMENTAL RESULTS

Fig. 3 shows the results of the perception threshold. The mean values are indicated by a solid line. The mean values show that the side-to-side perception thresholds are lower than the fore-and-aft perception thresholds at 1.0-3.0 Hz. However, at 4.0-6.0 Hz, the subjects are more sensitive to fore-and-aft motion than to side-to-side motion. Although horizontal vibrations were given in the test room, half of the subjects perceived a vertical vibration at 6.0 Hz fore-and-aft vibration. This result agrees with the guideline in ISO 2631 [6] where the equal sensitivity curve for whole-body vibration shows that at 1.0-3.15 Hz human sensitivity to horizontal vibration is higher than to vertical vibration, and at other frequencies sensitivity to vertical vibration is higher.

Fig. 4 shows perception histograms at 4.0 Hz. These are illustrations of the determination of the probability distribution of perception limits, where the perception limits were standardized using the mean value at each frequency, and the log-normal distributions were compared.

Fig. 5 shows the probabilistic perception limits corresponding to 2, 10, 50, and 90% values. The figure also shows ISO 2631 base curve [6, 7] and the perception limits in the lower frequency range for the fore-and-aft motion by Kanda, Tamura et al. [2]. With respect to the side-to-side motion, the ISO base curve at 2.0-6.0 Hz roughly corresponds to the 10% values of our results. As for the fore-and-aft motion, the 50% values at 1.0-2.0 Hz coincide with the 50% values for the low-frequency tests [2].

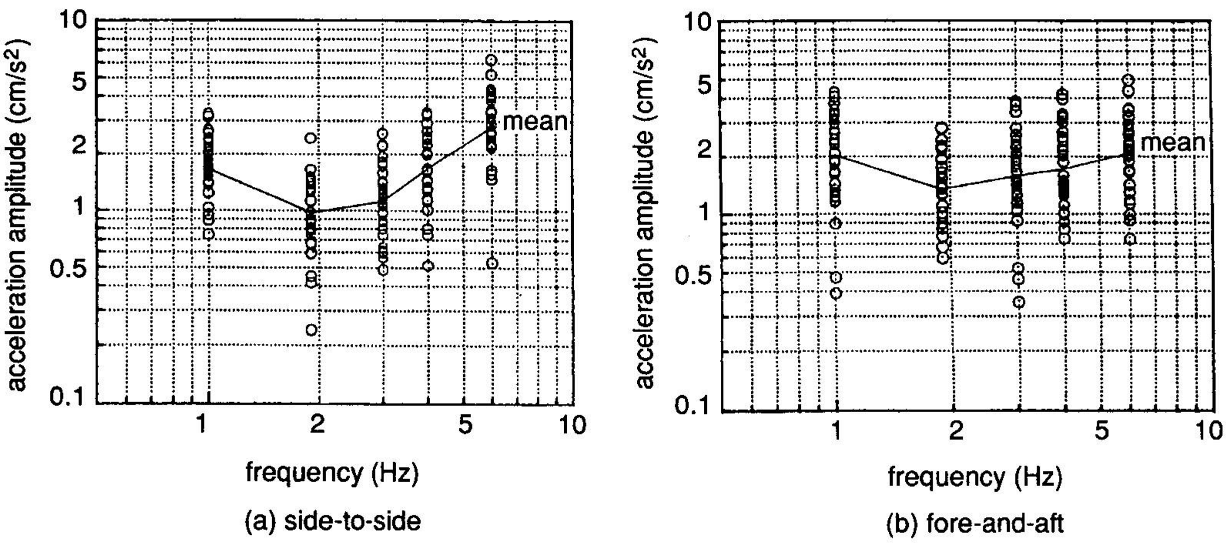


Fig.3 Perception limits

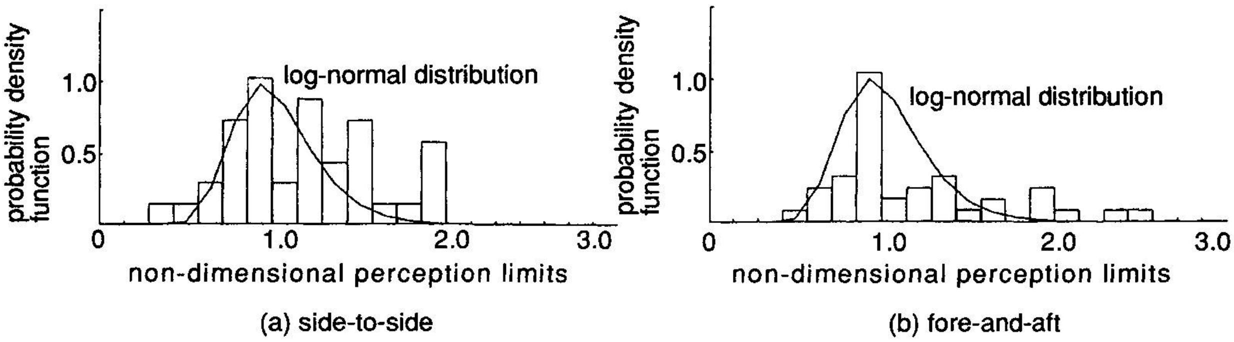


Fig.4 Examples of perception histograms (4.0Hz)

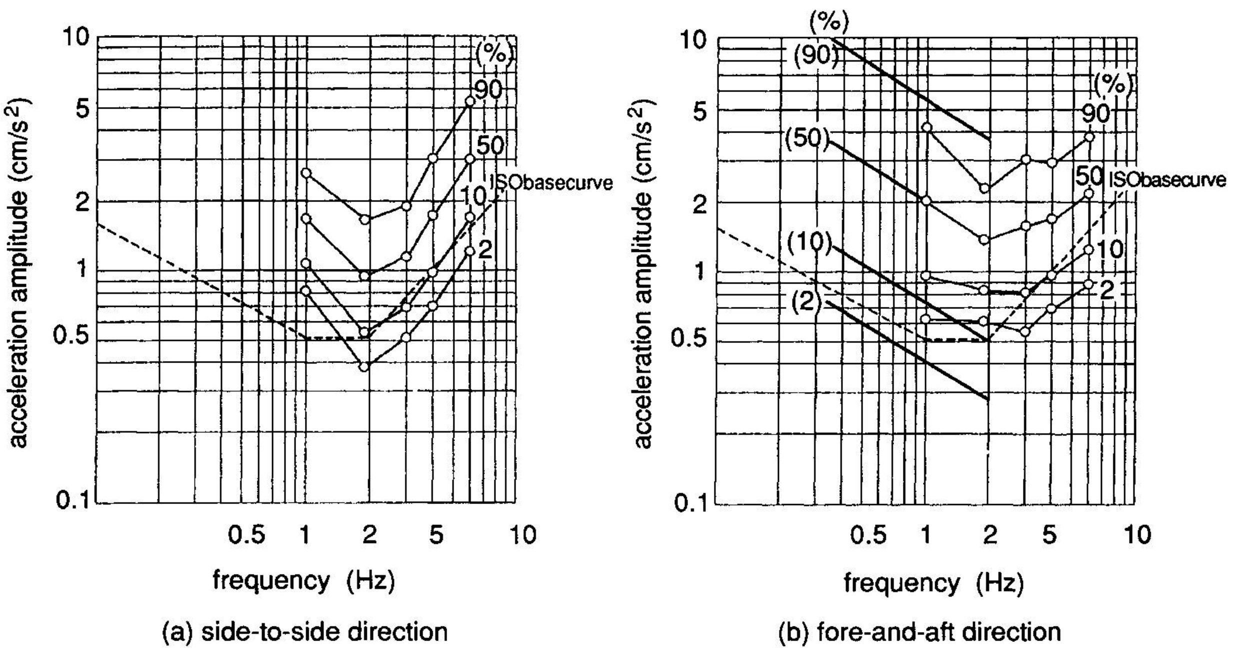


Fig. 5 Probabilistic perception limits and the comparison with other criteria and reports

4. CONCLUDING REMARKS

From the perception tests, the following conclusions were obtained:

(1) Perception limits vary with the frequency of motion and their individual variations are significant.

(2) The gradient of perception limits with the vibration frequency changes at around 2.0 Hz.

(3) The gradients of the ISO base curve and our results are almost consistent with each other.

ACKNOWLEDGEMENT

We would like to express our heartful thanks to Mr. Kunio Fujii of Wind Engineering Institute Co., Ltd. and Mr. Shunichi Naito of TAKU Architecture and Structural Engineering Co., Ltd. for their invaluable advice in connection with this study. Additionally, we would like to express our gratitude to Prof. Kanda and his colleagues for their research in this area of study.

REFERENCES

1. KANDA, J., TAMURA, Y., and FUJII, K., Probabilistic Criteria for Human Perception of Low-Frequency Horizontal Motions, Symposium/Workshop on Serviceability of Buildings (Movements, Deformations, Vibrations), 1988, Vol. 1, pp. 260-269.
2. KANDA, J., TAMURA, Y., and FUJII, K., Probabilistic Perception Limits of Low-Frequency Horizontal Motions, Conference with International Participation Serviceability of Steel and Composite Structures, 1990.
3. Committee on Allowable Limits of Vibration in High-Rise Housing, Study on Perception of Low-Frequency Vibration in High-Rise Housing—Part 1, 1988.
4. Study on the Influence of Motions and Vibrations in High-Rise Buildings on Habitability, 1983.
5. Architectural Institute of Japan, Guidelines for the Evaluation of Habitability to Building Vibration, 1991.
6. ISO, Evaluation of Human Exposure to Whole-Body Vibration—Part 2: Evaluation of Human Exposure to Vibration and Shock in Buildings (1 to 80 Hz), ISO/DIS 2631/2.
7. ISO, Guidelines for the Evaluation of the Response of Occupants of Fixed Structures, Especially Buildings and Off-Shore Structures, to Low-Frequency Horizontal Motion (0.063 to 1 Hz), ISO 6897, 1984.

Human Perception Thresholds of Horizontal Motion

Seuils de la perception humaine du mouvement horizontal

Menschliche Wahrnehmungsschwellen von horizontalen Schwingungen

Kiyoto SHIOYA
Senior Research Eng.
Shimizu Corp.
Tokyo, Japan

Jun KANDA
Prof. Dr.
University of Tokyo
Tokyo, Japan

K. Shioya, born 1949, M Eng., Dep. Architecture, Kyushu University, Fukuoka, Japan.

J. Kanda, born 1947, M Eng., Dep. Architecture, University of Tokyo Ph.D., Edinburgh University, Structural Engineer in Takenaka Komuten till 1980.

SUMMARY

Horizontal responses of tall buildings to wind are generally a narrow-band random process caused by the light damping of structures. It is necessary to evaluate human perception of motion under conditions as similar as possible to those that exist in tall buildings. The human perception thresholds of a narrow-band random process at predominant frequencies of 0.125 Hz to 0.315 Hz are examined and compared with those of sinusoidal motion. Statistical analyses were performed using the experiment data to propose probabilistic criteria for the perception thresholds.

RESUME

La façon dont les bâtiments réagissent au vent est généralement dans des limites étroites, influencées par un léger amortissement des structures. Il est nécessaire d'évaluer la perception humaine sous des conditions aussi semblables que possibles à celles qui existent dans les bâtiments. Les seuils de la perception d'une toute petite action horizontale inattendue, à fréquences prédominantes de 0.125 Hz à 0.315 Hz, sont examinés et comparés avec ceux d'un mouvement sinusoïdal. Les analyses statistiques ont été établies en utilisant les données expérimentales pour proposer des critères probables pour les seuils de la perception.

ZUSAMMENFASSUNG

Horizontale Antworten hoher Gebäude auf Wind sind normalerweise Zufallsereignisse innerhalb eines schmalen Bandes, bedingt durch die leichte Dämpfung des Tragwerks. Es ist notwendig, die menschliche Wahrnehmung der Schwingungen unter Bedingungen zu bewerten, die so ähnlich wie möglich zu denen sind, die sich in hohen Gebäuden finden. Die menschlichen Wahrnehmungsschwellen von engbandigen zufälligen Vorgängen bei vorherrschenden Frequenzen von 0.125 Hz bis 0.315 Hz werden untersucht und denen bei Sinusschwingungen gegenübergestellt. Statische Analysen wurden unter Verwendung der Experimentdaten durchgeführt, um probabilistische Kriterien für die Wahrnehmungsschwellen vorzuschlagen.

1. INTRODUCTION

Horizontal responses of tall buildings to wind are generally a narrow-band random process caused by the light damping of structures. The fundamental natural frequencies of structures are dominant in the time histories of responses. The traces of response are elliptical in shape. It is necessary to evaluate human perception of motion under conditions as similar as possible to those that exist in tall buildings. Although investigations have been conducted on the human perception of horizontal vibration in existing tall buildings[1],[2], the relationship between the perception of random and sinusoidal motion has not been established.

The authors first studied the human perception thresholds of uniaxial, elliptical and circular sinusoidal motions to discuss the effect of the two-dimensional motion[3]. Secondly, we studied the perception thresholds of a narrow-band random process to investigate the effect of random motion.

In this paper, the human perception thresholds of a narrow-band random process at predominant frequencies of 0.125Hz to 0.315Hz are examined and compared with those of sinusoidal motion. Statistical analyses were performed using the experiment data to propose probabilistic criteria for the perception thresholds.

2. TEST METHOD

2.1 Testing Condition

Random motions were synthesized by calculating the response of a single degree of freedom system with light damping (h = 0.01) to the Gaussian white noise with uniformly distributed random phase. Random motions contained combinations of the following parameters: Predominant frequencies - 0.315Hz, 0.25Hz, 0.2Hz, 0.16Hz, 0.125Hz, Body orientations - fore and aft (X direction), side to side. (Y direction). Figure 1 shows the time history of a typical narrow-band random process.

People in the test were placed in a sitting position to the horizontal movements. Human reactions were classified in three ratings as follows : [A] Imperceptible ; [B] Barely perceptible (Level I) ; [C] Distinctly perceptible (Level II). Each subject showed his/her rating of each motion by pushing one of three buttons. A total of 61 people were tested in this experiment.

2.2 Testing Equipment

The equipment used as a vibration generator was the electro-hydraulic servo-controlled 6 degrees of freedom shaking table. A testing room of 3.1m × 4.0m with a 2.6m ceiling was mounted on the shaking table with laminated rubber bearings to cut off higher frequency motions of the shaking table. To avoid the subjects being influenced by noise from the actuator, acoustical insulation was installed in the walls and ceiling

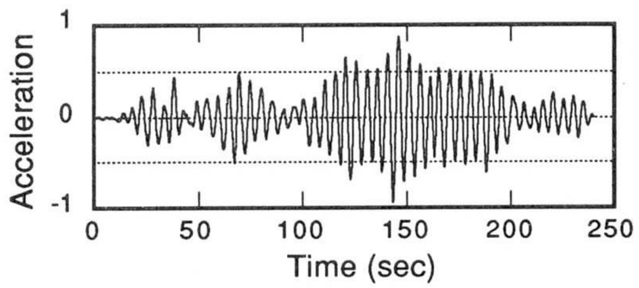


Fig.1 Synthesized Random Process

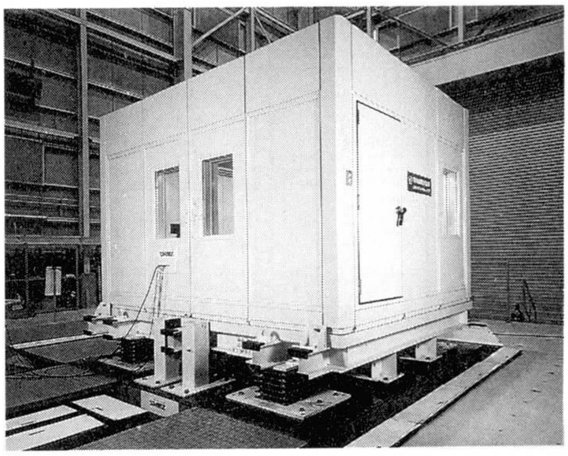


Fig.2 Vibration Simulator

of the testing room. The blinds were pulled down during the measurement so that the subjects could not perceive the motion of the room by sight. Highly sensitive servo-controlled accelerograms were used in three different directions to measure the motion of the vibration simulator.

The exterior of the vibration simulator and body orientation are shown in Figures 2 and 3.

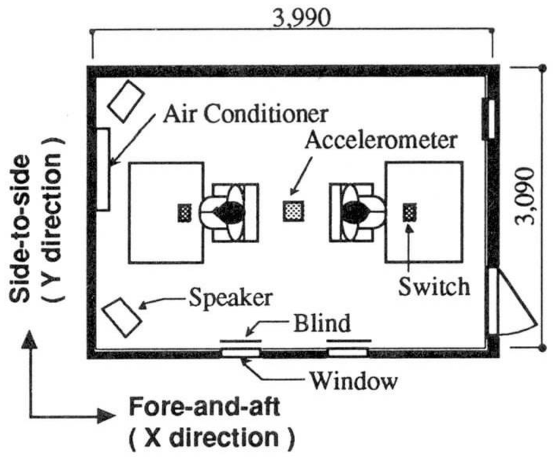


Fig.3 Body Orientation

3. PERCEPTION THRESHOLDS OF RANDOM MOTION

3.1 Detection of Perceived Acceleration Level

The subject showed the rating of each motion by pushing buttons. The perceived accelerations of each subject were read at the switching points of the absolute acceleration envelope in a positive gradient. The perceived accelerations were obtained at several points because the motion was a random process. The mean values of each case were used as his/her perception values.

The relationship between the frequency and the perceived acceleration of all cases was shown in scattered plots and the mean values and standard deviations were calculated by using the data of rating [B] and [C]. The scattergrams of perceived acceleration are shown in Figure 4 . The solid line shows mean values and the dotted line shows standard deviations of each frequency.

The coefficients of variation (COV) are 0.31 to 0.72 in Level I and 0.21 to 0.42 in Level II. It was found from the scattergrams that the dispersion of Level II is smaller than that of Level I and that the perceived accelerations are almost independent of the frequency, especially in Level II in both motion directions.

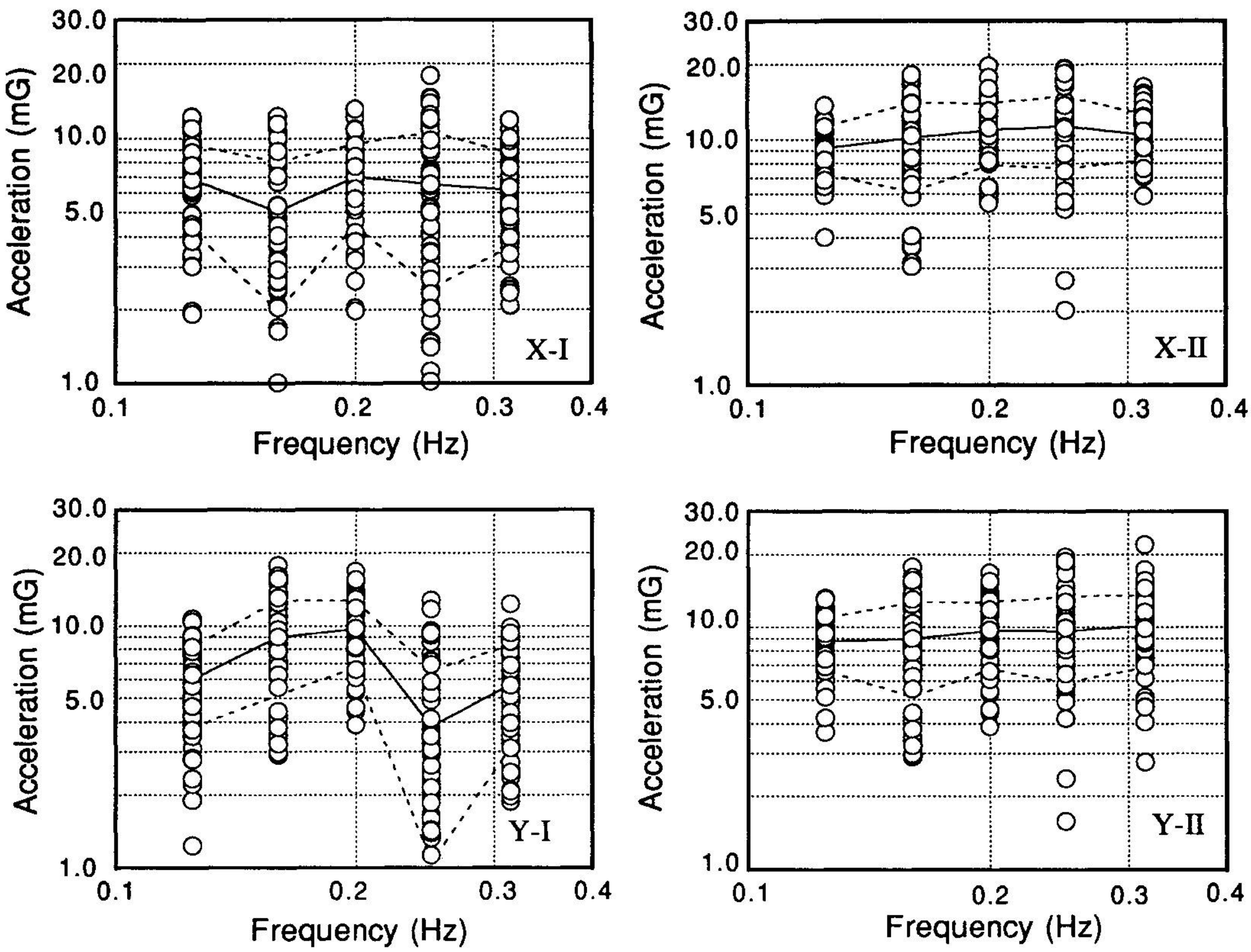


Fig.4 Scattergrams

3.2 Probability Density Function of Perceived Acceleration

Normalized probability density functions of perceived acceleration were calculated by dividing data (X_{ij}) by the mean value of corresponding cases ($X_{j,mean}$). Figure 5 gives the probability density functions of all cases.

The normalized probability density functions were examined by the Chi-square test for the goodness of fit to the normal distribution and the log-normal distribution. It was found that both distributions fit the present data within a 5% significance level, but the log-normal distribution gave a closer approximation. After the Chi-square test and the non-negative of perception level, the log-normal distribution was better than the normal distribution as the probability distribution model of perception threshold.

3.3 Percentile of Perception

The log-normal probability density function is assumed as the model of perception threshold. The log-normal probability density functions of each frequency were first calculated then the probability distribution functions were obtained. The 2, 10, 50 and 90 percentile perception levels were calculated from the distribution functions.

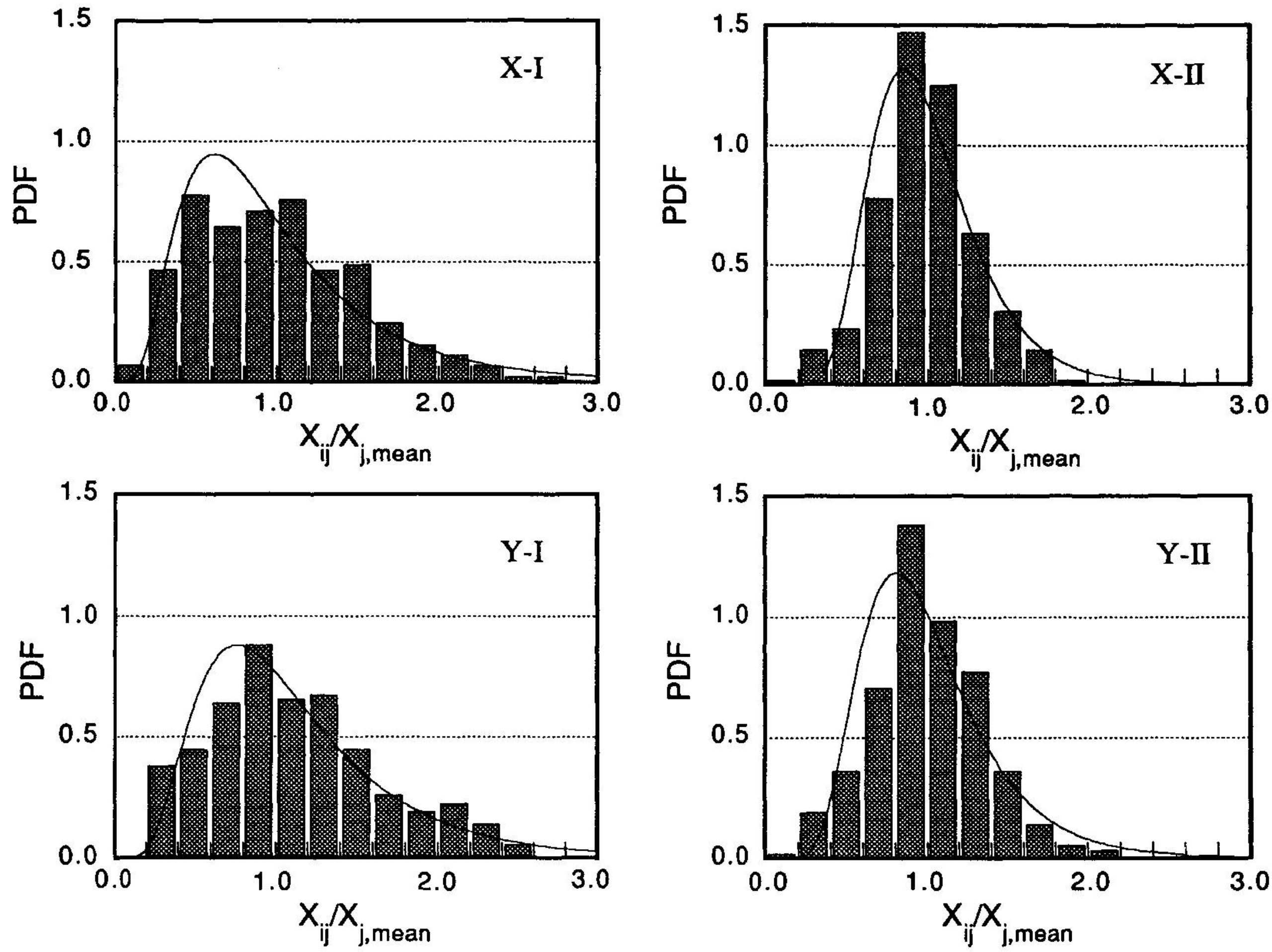


Fig.5 Probability Density Function of Perception

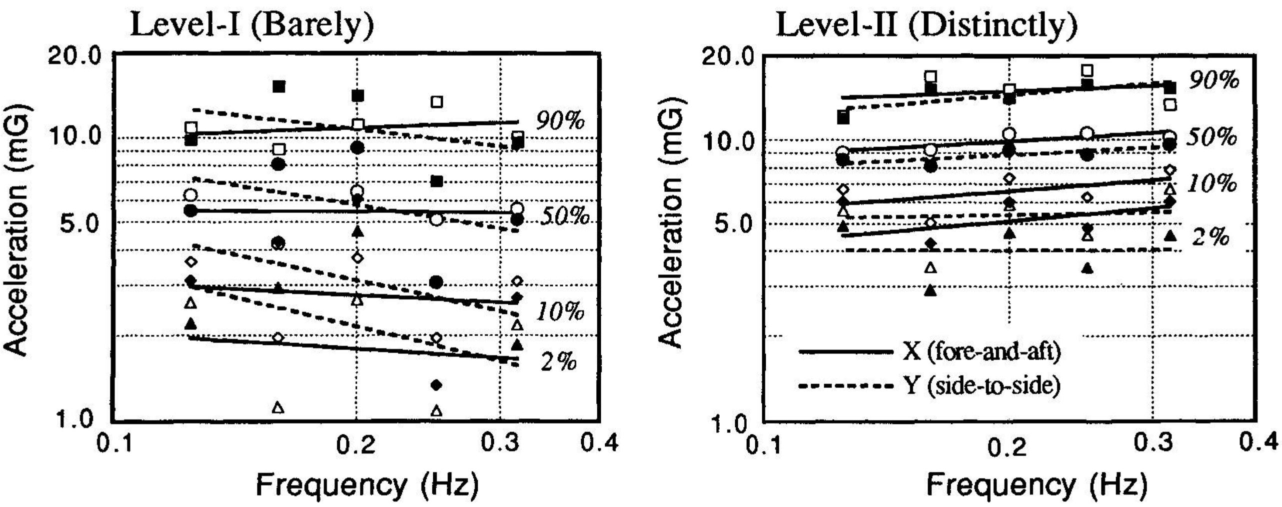


Fig.6 Percentile of Perception to Random Motion

Regression analyses were made on each percentile perception level in the logarithmic scale. Figure 6 shows the regressed percentile of perception of all cases.

The perception dispersion of Level I is rather larger than that of Level II. As the correlation coefficients between perceived acceleration and frequency are very low in

Level I, we examined the result of Level II only.

The side-to-side (Y direction) motions are rather more perceptible than the fore-and-aft (X direction) motions.

The gradient of regression lines are positive and the absolute value of the gradient is very small. The perception levels of random motions are almost constant to the frequency.

The regressed lines of percentiles are parallel to each other. The 50 percentile lines of Level II are nearly equal to 0.01 G.

4. COMPARISON WITH THE SINUSOIDAL MOTION

4.1 Perception of Sinusoidal Motion

The authors studied the perception thresholds of uniaxial, elliptical and circular sinusoidal motions to discuss the effect of the two dimensional motion in the same manner as presented in this paper[3]. We proposed the probabilistic model of perception dispersion. The results of the sinusoidal test are described as follows.

(1) The side-to-side (Y direction) motions are rather more perceptible than the fore-and-aft (X direction) motions in most cases.

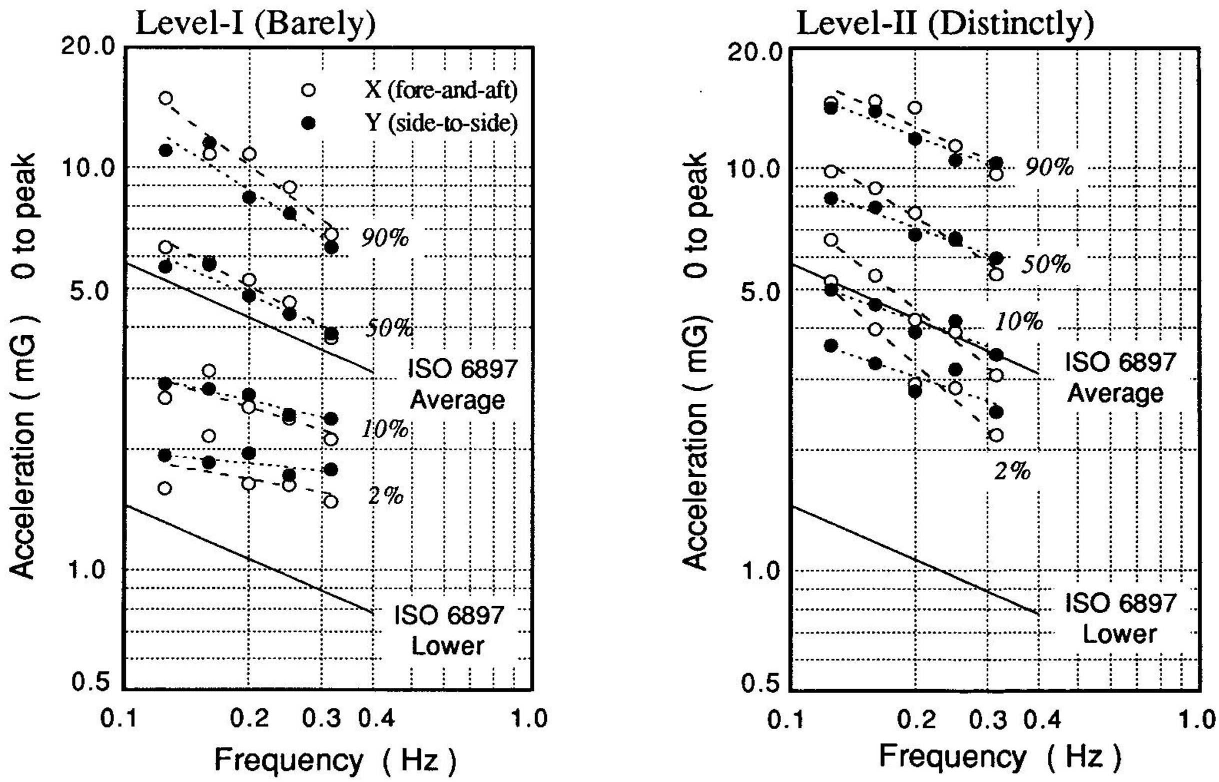


Fig.7 Percentile of Perception to Sinusoidal Motion

(2) The perceived accelerations of the uniaxial motions are almost the same as those for the elliptical and circular motions.

(3) According to the questionnaire studies, half the subjects could not distinguish between uniaxial and elliptical motion.

(4) The log-normal distribution fits the probability distribution model of perception threshold.

(5) The 50 percentile regression lines of Level I agreed approximately with the ISO average threshold line in all cases.

(6) The regression lines of Level II were approximately 20% higher than those of Level I in all cases.

The 2 - 90 percentile of perception thresholds of sinusoidal motion are shown in Figure 7.

4.2 Comparison with Sinusoidal Test

As the perception dispersion of Level I was too large, the 50 percentile lines of the perceived acceleration of random motion of Level II were compared with that of sinusoidal motion. Figure 8 gives the results of the comparison. The average threshold lines of the ISO 6897 are given in the figures for reference.

The 50 percentile thresholds (Level I) of sinusoidal motions agreed approximately with the ISO average threshold line.

The gradient of the regression lines for the sinusoidal motions are negative to the frequency.

On the other hand, the gradient of the regression lines for the perception levels of random motions are positive and almost constant to frequency. This may be due to the subjects' delay of response to vibrations in higher frequencies. The 50 percentile lines of Level II are nearly equal to 0.01G.

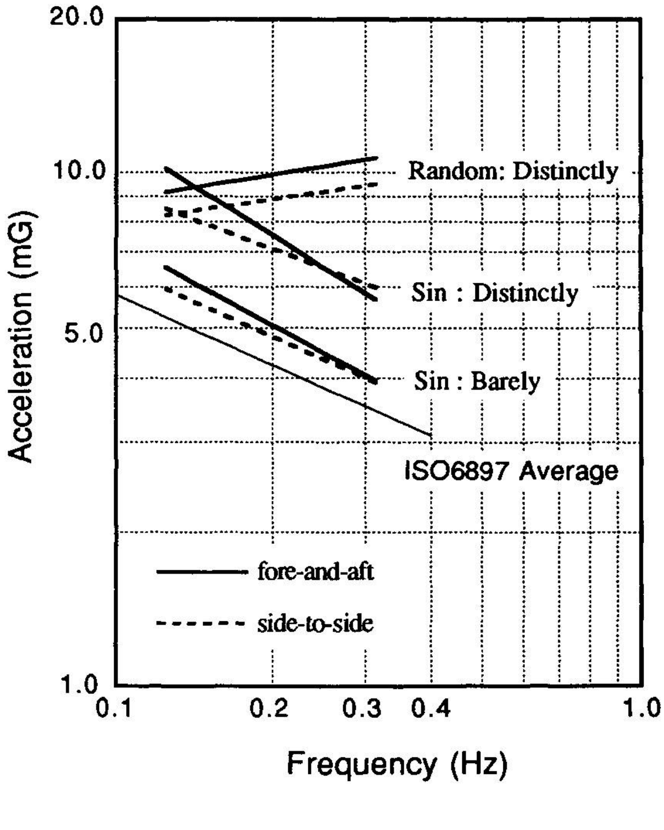


Fig.8 Comparison with Sinusoidal Test

5. SUMMARY AND CONCLUSIONS

The perception thresholds of the narrow-band random motions synthesized as the response of tall buildings to wind were obtained within a highly significant level in the frequency band 0.315 Hz to 0.125 Hz.

(1) The side-to-side (Y direction) motions are rather more perceptible than the fore-and-aft (X direction) motions.

(2) The log-normal distribution fits the probability distribution model of perception threshold.

(3) The perception dispersion of the barely perceptible level (Level I) is rather larger than that of the distinctly perceptible level (Level II).

(4) The perceived acceleration of Level II is almost constant to the frequency.

(5) The 50 percentile lines of Level II are nearly equal to 0.01 G.

Acknowledgments

Grateful acknowledgment is given to Professor Y. Tamura, Tokyo Institute of Polytechnics, Dr. K. Fujii, Wind Engineering Institute Co.,Ltd., Professor T. Goto, Hosei University, and Professor T. Ohkuma, Kanagawa University, who gave valuable assistance in this work.

References

1. HANSEN, R. J. , REED, J. W., and VANMARCKE, E. H., Human Response to Wind-Induced Motion of Buildings. J. Struct. Div. ASCE, Vol.97, No.ST7, July 1973

2. JEARY, A. P., MORRIS, R. G., and TOMLINSON, R. W., Perception of Vibration - Tests in a Tall Building. Journal of Wind Engineering and Industrial Aerodynamics, Vol.28, 1988

3. SHIOYA, K., KANDA, J., TAMURA, Y., and FUJII, K. , Human Perception Thresholds of Two Dimensional Horizontal Motion. ASCE Structures Congress '92, April 1992

4. ISO 6897 : Guidelines for the evaluation of the response of occupants of fixed structures, especially buildings and off-shore structures, to low-frequency horizontal motion (0.063 to 1 Hz)

Human Response to Horizontal Motion of Tall Buildings

Réaction humaine aux mouvements horizontaux des gratte-ciels

Menschliche Reaktion auf Horizontalschwingungen in Hochhäusern

Kenichi NOGUCHI
Research Engineer
Taisei Corporation
Tokyo, Japan

Kiyoshi HIWATASHI
Research Engineer
Taisei Corporation
Tokyo, Japan

Akio KOBAYASHI
Research Engineer
Taisei Corporation
Tokyo, Japan

Osamu TSUJITA
Engineer
Taisei Corporation
Tokyo, Japan

Takeshi GOTO
Professor
Hosei University
Tokyo, Japan

SUMMARY

The authors have conducted experiments on human response to typical horizontal biaxial motions in order to clarify the effects of the motions of tall buildings induced by wind on habitability. The tests proved the following by the experience of discomfort, difficulty and uneasiness and the balance of standing persons: Human response to biaxial motions, especially to circular and linear motion, is much greater than that to linear motion; and, the relation between balance shift index and psychological response can be approximated by one logarithmic function.

RESUME

Les auteurs ont mené des essais sur le comportement humain par suite de mouvements horizontaux biaxiaux, engendrés par l'effet du vent sur les immeubles de grande hauteur. Il en résulte que les humains réagissent plus fortement aux mouvements biaxiaux, tout spécialement à la combinaison des mouvements circulaires et de roulis, qu'aux mouvements uniquement linéaires. Les essais ont montré que le déplacement du centre de gravité de l'être humain concorde de manière quasiment logarithmique avec sa réaction psychologique. Les degrés d'inconfort, d'inquiétude et de difficulté à conserver l'équilibre ont servi à chiffrer cette évaluation.

ZUSAMMENFASSUNG

Versuche zur menschlichen Reaktion auf typische Horizontalbewegungen in beiden Achsrichtungen in Hochhäusern haben ergeben, dass Menschen stärker auf biachsiale, Kreisel- und Schlingerbewegungen reagieren als auf Linearschwingungen und dass die Schwerpunktsverlagerung fast logarithmisch mit der psychologischen Reaktion übereinstimmt. Zur Bewertung wurde dabei das Ausmass des Unbehagens, der Beklemmung und der Gleichgewichtsstörungen verwendet.

1. INTRODUCTION

The history of tall building construction began with an encounter of our admiration for height with advanced technology. Authors of this paper have been engaged in the development of a 100 story multi-purpose building which has dwelling floors on its upper stories. The building is 480 meters high and has a high aspect ratio, therefore, the upper part of the building can be subject to complex and large motion with long period when strong wind blows. Accordingly, the vibrations give some influence on the serviceability, especially habitability of the rooms in the building.

There are guidelines[1) 2)] for the evaluation of the human response to horizontal motion provided by the International Organization for Standardization (referred to as ISO for the rest of this paper) and the Architectural Institute of Japan (referred to as AIJ for the rest of this paper). These guidelines suggest that tests should be performed on threshold of perception to horizontal uniaxial motion. Whereas, tests performed in this manner are thought to be inadequate because tall buildings are subject to complex horizontal motion in actuality.

In this paper, tests are conducted on human response to horizontal biaxial motion using a motion simulator. In the tests, the psychological response of subjects was surveyed by conducting questionnaire and the physical response of subjects was obtained by measuring the balance shift of the subjects. After obtaining test data, habitability to horizontal biaxial motion is studied on the basis of available data on those with horizontal uniaxial motion. Then the test results are compared with existing guidelines, and the correlation between physical response and psychological response is studied.

2. PLANNED BUILDING AND MOTION

The planned building is 100 storied and 480 meters high. It is composed of vertically dissectioned four blocks (see Fig.1) so that vibrations caused by strong wind are attenuated and structural safety is ensured.

According to the model test and analysis, the top of the planned building causes largest displacement to the wind direction of 0°, when response wave forms indicate an extreme motion around the first natural period (8.0sec) of the building. Fig.2 is the locus of the top displacement of the building when the wind velocities with a return period of one year (level 0), and those with a return period of 100 years (level 1), are applied in Tokyo. When the wind velocities of level 1 are applied, vibrations are extreme in the cross wind direction and show an elliptic response. When the wind velocities of level 0 is applied, vibrations show a circular response.

For the reasons mentioned above, habitability can not be assessed adequately by human response to uniaxial motion, and human response to horizontal biaxial motion must be evaluated.

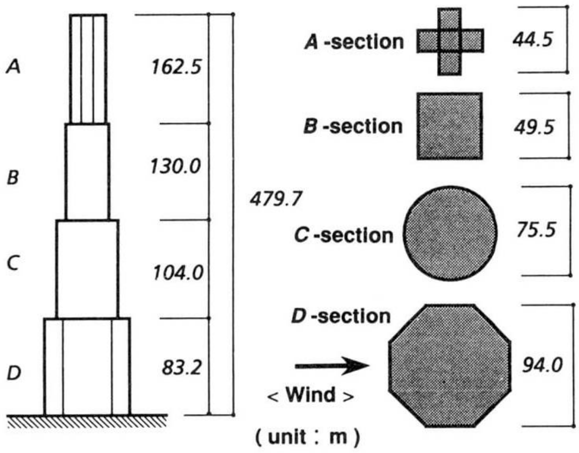


Fig.1 Planned building

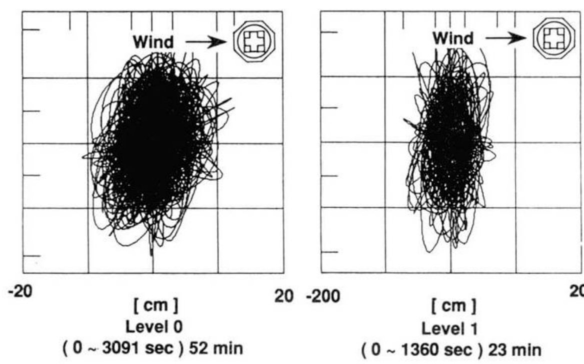


Fig.2. Locus diagram of top displacement of the building

3. HUMAN RESPONSE TEST

3.1 Test equipment

Tests are performed in a testing room (2.4m x 2.4m x H2.4m) installed on the six-degree of freedom motion simulator (in Hosei University, Tokyo, Japan) (see Fig.3).

3.2 Human subjects

Twenty subjects are chosen from female aged between 37 to 58. House wives, who most stay at home, are selected because a daily life environment is assumed in these tests.
Subjects are divided into groups of four members.

3.3 Conditions

Test conditions are determined so that acceleration amplitudes of motion are in accordance with ISO curve 1 (suggested satisfactory magnitudes of horizontal motion of building used for general purposes) and AIJ H-4 curve. Vibration period is set to five to ten seconds, which covers the first natural period of the planned building. Motion types are linear, elliptic, circular, and eight-figure, each of which has sinusoidal motions. Detailed conditions are given in Table 1. The order of motion conditions for tests are decided using a table of random numbers and so that each group of subjects has a different order.

3.4 Procedure

The testing room has no window so that subjects can not visually perceive vibrations. Subjects stand still with their eyes opened in the testing room. Each motion lasts for ten minutes. After one minute from the start of each test, balance shifts of the subjects are measured for one minute, and after five and ten minutes, they are requested to vote each item of discomfort, difficulty, and uneasiness they perceived on five levels. The category scale is shown in Fig.6.

4. VIBRATION AND BALANCE SHIFT

Fig.4 is the typical locus curve of balance shift of subject with each motion type. In all cases, balance shift indicates the locus similar to the motion type. It should be noted that a half moon shaped locus in the case of eight-figure motion is due to phase shift of X direction and Y direction.
Fig.5 shows the balance shift (r.m.s. value) of human subjects in each of the X direction and Y direction. Values are the average of all

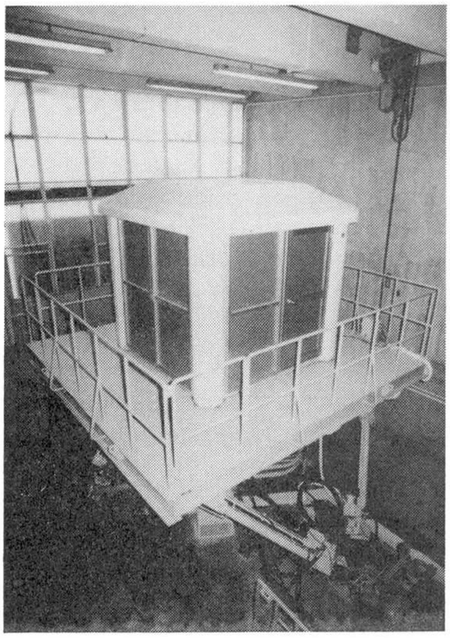


Fig.3 Motion simulator

Table 1 Motion conditions of the test

Figure of Motion	Period (sec)	Maximum Acceleration of X-direction (cm/s^2)	Ratio of Maximum Acceleration of two direction : RXY 1)
Linear	10.0	9.1～14.2	0.0
	8.0	8.0～12.3	
	6.0	6.6～8.8	
	5.0	4.7～7.9	
Circular	10.0	9.1～14.2	1.0
	8.0	8.0～12.3	
	6.0	6.6～8.8	
	5.0	4.7～7.9	
Elliptic	10.0	9.1～14.2	0.5
	8.0	8.0～12.3	
	6.0	6.6～8.8	
	5.0	4.7～7.9	
Eight - figure 2)	10.0	9.1～14.2	0.2
	8.0	8.0～12.3	
	6.0	6.6～8.8	
	5.0	4.7～7.9	

1)Ratio of Maximum Acceleration of two direction

$$RXY = \frac{\text{Maximum Acceleration of Y-Direction}}{\text{Maximum Acceleration of X-Direction}}$$

2)Period of Y-Direction is half for that of X-Direction

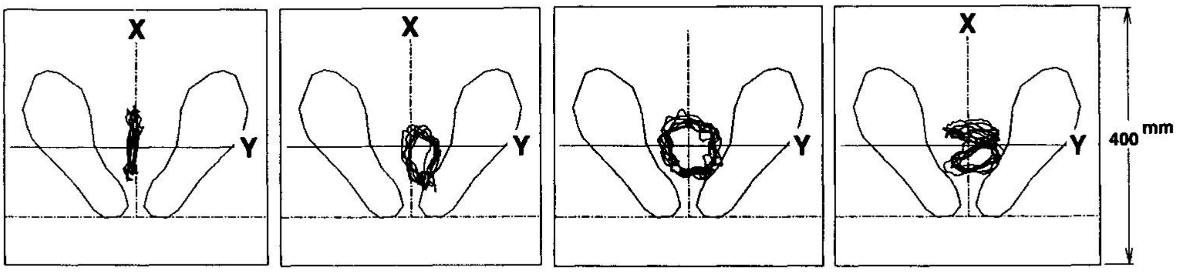


Fig.4 Locus diagram of balance shift of subjects

subjects.
Each motion type(linear, elliptic, circular, eight-figure) has the same amplitude of acceleration in the X direction but the different amplitude of acceleration in the Y direction. Therefore, balance shift shows little difference in the X direction but considerable difference in the Y direction with the motion type.

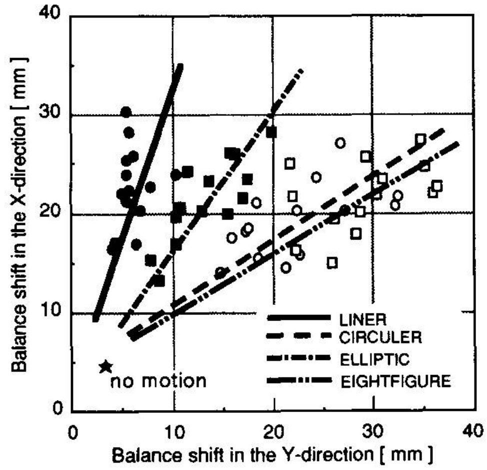


Fig.5 Balance shift in X/Y directions

5. VIBRATION ACCELERATION AND VOTINGS OF PSYCHOLOGICAL RESPONSES

When the voted value obtained after 5 minutes and that 10 minutes are compared, the later is slightly lower than the former. This may be caused by the habituation of the subjects. Hereafter, the values obtained after 5 minutes are used for examination.
Biaxial motions, especially circular and eight-figure motions, have more influence on human response than linear motion does, by about 0.6~0.8 rank in the five level voting system in this paper. This is because circular and eight-figure motions cause balance shift in both Y and X directions while linear motion causes balance shift mainly in the X direction as shown in Fig.4 and 5.
The relationship between the vibration acceleration amplitude and voting of psychological response can be approximated by the Weibull curve as shown in Fig.6. The vibration acceleration amplitude which corresponds to a certain voting level at each vibration frequency can be obtianed from using this relationship.
In Fig.7, the vibration acceleration amplitude in the X direction which corresponds to the voting of 2.5 is shown together with ISO curve 1 and AIJ H-4 curve.
The voting of 2.5 refers to the level between "slightly perceptible" and "clearly perceptible".
ISO curve 1 is the criterion that probably not more than 2% of those occupying the parts of the building where the motion is greatest comment adversely about the motion caused by the peak 10 minutes of the worst wind storm with a return period of 5

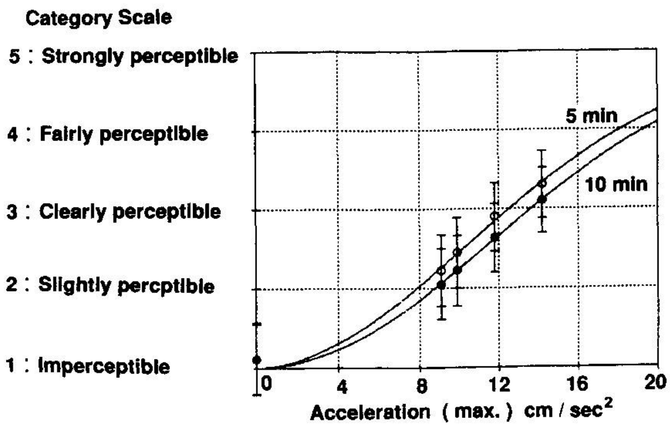


Fig.6 Vibration acceleration amplitudes and votings

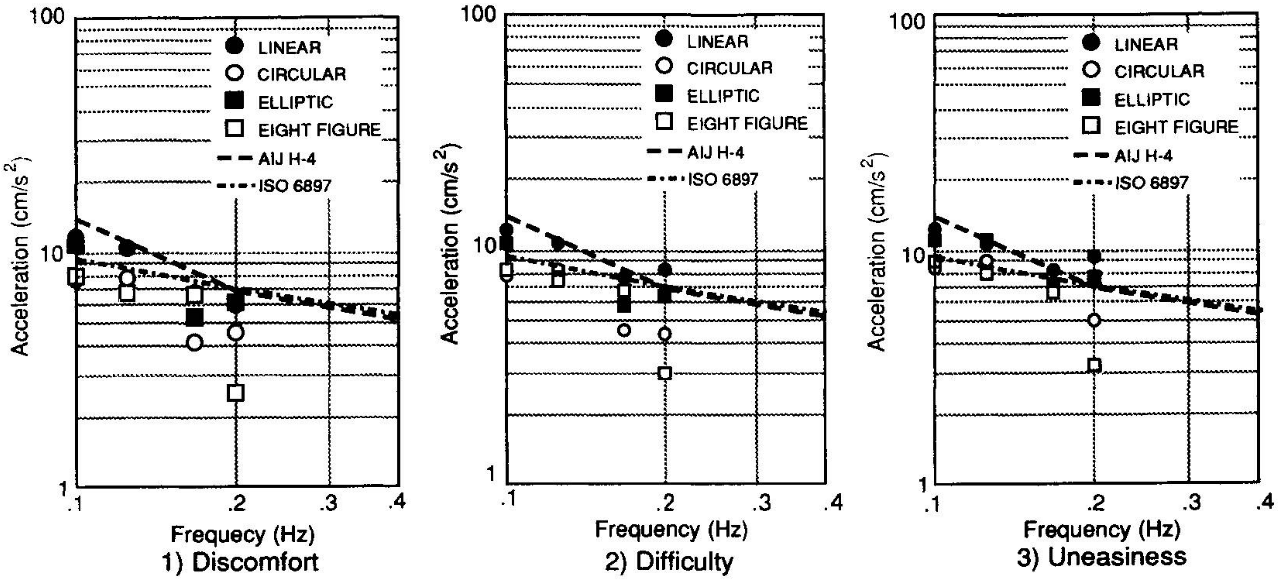


Fig.7 Relationship between vibration acceleration amplitudes rated 2.5 and existing guidelines

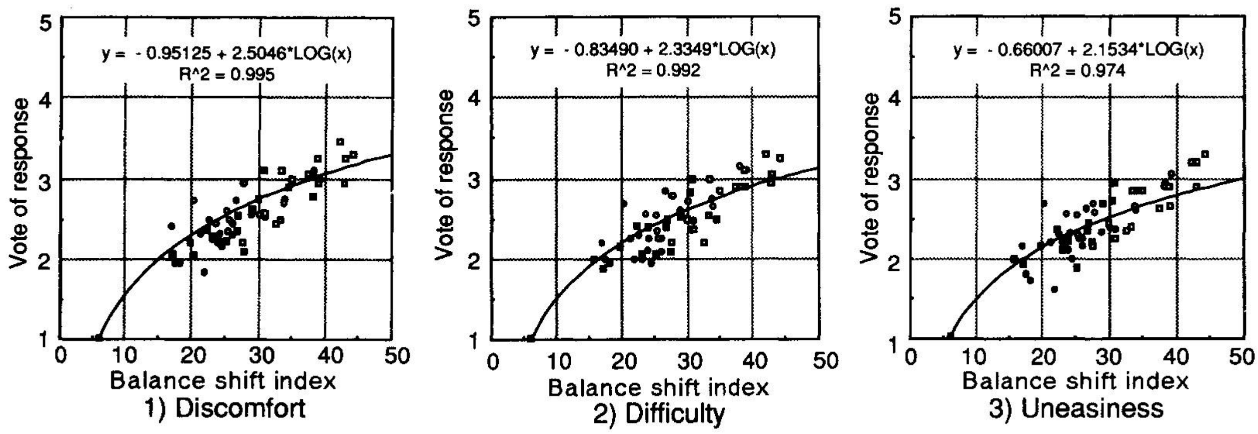


Fig.8 Balance shift indices and votings

years or more. AIJ H-4 curve is 1.75 times of the average threshold of perception obtained by human response tests to sinusoidal motions. Complaint categories dealt with in this paper, discomfort, difficulty, and uneasiness, can not be compared absolutely but it is possible to compare them relatively.

The vibration acceleration amplitudes which received the psychological voting of 2.5 almost agree with ISO curve 1, but there exist slight differences with the 3 items above.

However, a ratio of the increasing perceptible acceleration amplitude to the decreasing frequency, namely the incline of the curve, is similar to AIJ H-4 curve. Comparing with the complaint categories, perceptible acceleration amplitude increases in order of discomfort, difficulty, and uneasiness with all motion types. That means subjects feel discomfort before feeling difficulty, and feel difficulty, before feeling uneasiness. This result is in line with authors' expectation, however, it should be noted that subjects do not feel uneasiness so much because they know it is a test.

6. BALANCE SHIFT AND VOTINGS

Next, psychological response (ratings) is considered in relation to balance shift. Fig.8 shows the relationship between the balance shift index and voting. The

balance shift indices are the square root of the variance of balance shift in both X and Y direction. With all complaint categories, the balance shift index and voting decrease in order of linear, ellipse, circular, and eight-figure motion. The relationship between the balance shift index and voting can be approximated by one logarithmic function for all motion types, and used for explaining the relationship between physical and psychological response to biaxial motion.
Comparing the voting with balance shift index, the voting decreases in order of discomfort, difficulty, and uneasiness with a certain balance shift index.

7. CONCLUSION

Tests were conducted on human response to typical horizontal biaxial motion. The psychological response of subjects was surveyed by conducting questionnaire and the physical response of subjects was obtained by measuring the balance shift of the subjects.
Comparison of the test results with existing guidelines for the evaluation of habitability and study on the relationship between balance shift and psychological response brought about the following conclusions:

(1) The vibration acceleration amplitude which received the voting of 2.5 (the level between "slightly perceptible" and "clearly perceptible") for all cases of discomfort, difficulty, and uneasiness agrees with perceptible acceleration amplitude ISO curve 1. But, a ratio of the increasing perceptible acceleration amplitude to the decreasing frequency, namely the incline of the curve, is similar to AIJ H-4 curve.
(2) With all motion types, biaxial motions increase in order of discomfort, difficulty, and uneasiness. This is in line with authors' expectation though it might be difficult for subjects to feel uneasiness because of the test settings.
(3) Horizontal human response, especially circular and eight-figure, have more influence on humans than horizontal linear motion does, by about one rank in the five level voting system in this paper.
(4) The relationship between the balance shift index which is the square root of the variance of balance shift levels in both X direction and Y direction and voting can be approximated by one logarithmic function for all motion types, and used for explaining the relationship between physical and psychological response to biaxial motion.

REFERENCES

1) ISO 6897-1984(E) Guidelines for the evaluation of the response of occupants of fixed structures, especially buildings and off-shore structures, to low-frequency horizontal motion (0.0063 to 1Hz)

2) Guidelines for the evaluation of habitability to building vibration: Part 2: AIJ Recommendations, 1991

3) T. Goto, Y. Tsuboi, K. Noguchi, N. Furukawa : Inspection tests of six-degrees of freedom motion simulator. Part 1 and Part 2, Summaries of Technical Papers of Annual Meeting of A.I.J.: Oct. 1986

4) T. Oohashi, O. Tsujita et al.: Evaluation of wind induced vibration for a skyscraper with non-uniform section. Part 1 and Part 2, Summaries of Technical Papers of Annual Meeting of A.I.J.: Sept. 1991

Probabilistic Criteria for Serviceability Limit of Wind Response

Critères probabilistes pour l'aptitude au service relative au vent

Probabilistische Gebrauchstauglichkeitskriterien für Windschwingungen

Jun KANDA
Assoc. Professor
University of Tokyo
Tokyo, Japan

Yukio TAMURA
Prof. Dr.
Tokyo Institute of Polytechnics
Atsugi, Japan

Osamu NAKAMURA
Engineering Consultant
Wind Engineering Institute
Tokyo, Japan

Kiyoshi UESU
Senior Res. Eng.
Housing and Urban Dev. Corp.
Tokyo, Japan

SUMMARY

Probabilistic criteria are proposed for serviceability limit state design for annual maximum wind speed. The variability of human perception limit based on simulated experiments is taken into account. Numerical examples for number of days per year of perception of motions are shown by utilizing wind speed data in Tokyo. Comparisons with an existing guideline in Japan are also made.

RESUME

Des critères probabilistes sont proposés pour l'état limite de l'aptitude au service pour la vitesse maximale, annuelle, du vent. La limite de perception humaine basée sur les expériences simulées est prise en considération. Des exemples numériques pour le nombre de jours par an pour la perception des mouvements sont indiqués en utilisant les données de la vitesse du vent à Tokyo. Des comparaisons avec une directive japonaise sont également effectuées.

ZUSAMMENFASSUNG

Es werden Wahrscheinlichkeitskriterien zur Bemessung auf Gebrauchstauglichkeit für jährliche maximale Windgeschwindigkeiten vorgeschlagen. Dabei werden die Unterschiede der menschlichen Wahrnehmungsgrenzen in simulierten Experimenten berücksichtigt. Numerische Beispiele bezüglich der Anzahl der Tage pro Jahr für die Wahrnehmung von Bewegungen werden unter Verwendung der Windgeschwindigkeitsdaten von Tokio gezeigt. Weiterhin werden Vergleiche zu bestehenden Richtlinien in Japan vorgestellt.

1. INTRODUCTION

Until recently design loads for most of tall buildings in Japan were dominated by the earthquake load rather than by wind load. Although the design criteria for the wind and the earthquake have to be carefully discussed on their relevance and consistency, wind responses of some flexible and/or light tall buildings, now, tend to be predicted in the range of deflection criteria. In such cases human perception of the wind response in fairly frequent storms can easily be anticipated.

Some vibration control devices have been installed in these buildings in order to suppress the wind response. However the serviceability criteria for wind responses are not well established. Several guidelines are available to describe the performance of the wind response, but they have not taken into account the variability of neither individual perception limits nor wind response magnitudes. And it is difficult for engineers to judge what is the appropriate level for the wind response.

Probabilistic criteria for the serviceability limit of wind responses are proposed based on some statistical information for the human perception of horizontal vibration. The meaning of the criteria is examined in various manners including comparisons with conventional criteria given to return period winds. Effects of the variability of wind response on the perception probability are also discussed.

2. SERVICEABILITY LIMIT STATE DESIGN

The magnitude of wind response is a function of the wind speed, which is a random variable. The perceptible level of motion varies considerably from a person to a person. Then a probability-based limit state design becomes necessary to take those variabilities into account. Since the perception of motion is a clear definition in comparison with the discomfort or the annoyance, it can be regarded as a representative serviceability limit for the wind response. Then a typical schematic flow of serviceability limit state design for wind-induced vibration is shown in Fig. 1.[1)]

Once a construction site is given, the probability distributiion of annual maximum wind speed is estimated from statistical meterological data. The magnitude of wind response can be calculated in several ways for buildings with known dynamic characteristics. For human perception problems the acrosswind response has to be estimated as its peak acceleration usually dominates that of the alongwind response. The estimation is made either by empirical formlae or by wind tunnel experiments.

The perception limit for horizontal sinusoidal motions, P, may be modeled by a log-normal distribution with the mean μ_p and the coefficient of variation (c.o.v.) which is assumend as 0.4 based on simulation experimants [2)], where μ_p in terms of the acceleration amplitude (m/s^2) is approximately expressed as follows,

$$\mu_p = 0.0148\, f_0^{-0.6} \tag{1}$$

where f_0 is the frequency (Hz) of motion. Eq.(1) was obtained in a frequency range between 0.3 Hz and 2Hz.

Then the probability of perception of motion, P_p, can be estimated. When the target reliability index β_T is given, the design format becomes,

$$P_p = \text{Prob}\,[\,A > P\,] \le \Phi(-\beta_T) \tag{2}$$

where A is the annual maximum response predicted in terms of the acceleration, and $\Phi(\cdot)$ is the standard normal cumulative distribution. Since the wind response is a random vibration, A must be a value equivalent to the amplitude of simusoidal motion to be compared with a ramdom variable P based on experiments. Tentatively $A=2\sigma_a$ is proposed[1], where σ_a is the standard deviation of random acceleration response.

When the probability distribution of A is also modeled by the log-normal, the reliability index β for non-perception, i.e., A<P, is obtained by a simple formula as,

$$\beta = \frac{\lambda_P - \lambda_A}{\sqrt{\varsigma^2_P + \varsigma^2_A}} \tag{3}$$

where $\lambda = \ln \mu - 1/2\,\xi^2$(mean of logarithm)

$\xi^2 = \ln\{\,1 + (\sigma/\mu)^2\}$ (variance of logarithm).

According to eq.(2) for design formula, β of eq.(3) is compared with β_T as showm in Fig.1, and $\beta \ge \beta_T$ concludes the procedure, while $\beta < \beta_T$ requires some changes in design parameters as indicated in the flow.

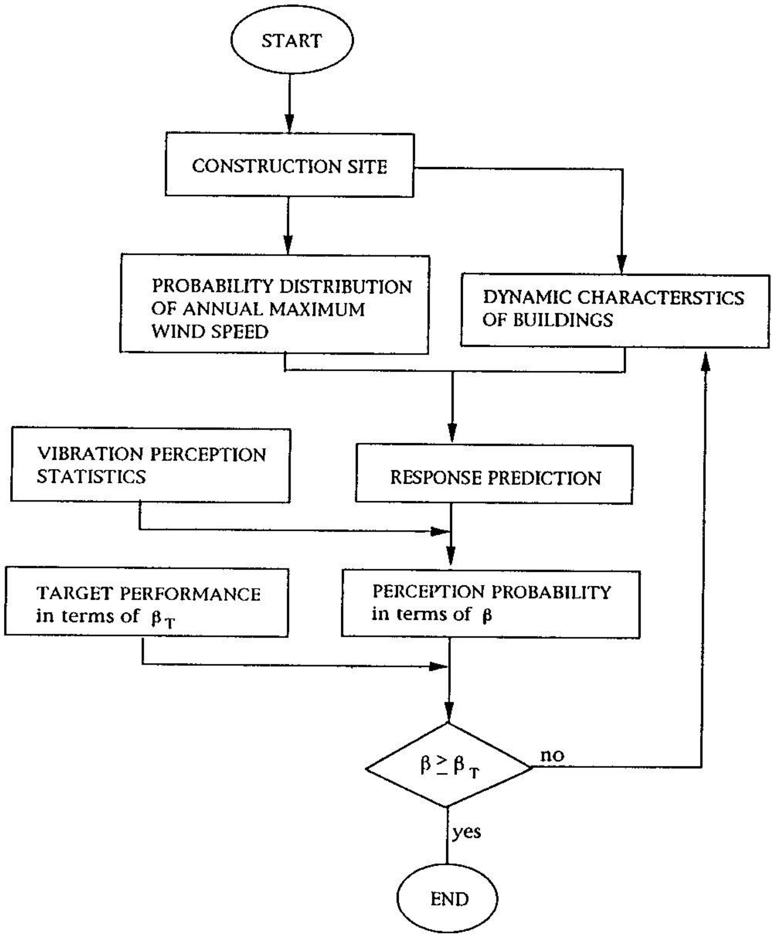


Fig.1. Schematic flow of serviceability limit state design for wind response

3. PROBABILISTIC CRITERIA FOR WIND RESPONSE

It is not so simple to judge which β is the most appropirate. Generally people do not expect that buildings vibrate in strong winds but they may accept if the perception of motion is not very frequent. Social and economical view points also have to be considered. Possible human reactions for β values may be described in Table 1.

Descriptions of human reactions are still only relative measures of performance. Nevertheless the β value provides objective wind response performance reflecting human perceptibility of motion. In order to discuss the appropriateness of β level, we need social experiments to examine feed-back opinions from occupants who are given information of design β. The annual probability of perception of motons is somehow general but rather abstract concept and can be explained more specifically in terms of number of days for peception of motions. Numerical examples are demonstrated for buildings in Tokyo area with different β values.

Wind data obtained at the Tokyo meteorological station during 10 years between 1979 and 1988 are utilized. The wind response in terms of equivalent acceleration amplitude , A, is assumed to be expressed as a function of U as,

$$A = \gamma U^{3.3} \tag{4}$$

When β is specified, γ is obtained for a frequency, f_0, in eq.(1) according to the serviceability limit state design described in the previous section with the coefficient of variation for both A and P as 0.4. According to the probability distribution model of P, wind reponse level A, at which 10%, 30% and 50% of occupants perceive the motions of frequency f_0, is calculated. From this A value for f_0, the corresponding wind speed U can be obtained by eq.(4) with γ representing β levels. Then the number of days when 10%, 30% and 50% of occupants perceive the motion can be counted for each year. This number is obtained as constant with the frequency f_0. Results are shown in Fig.2 for β=-1.0, 0.0 and 1.0.

Relatively frequent vibration perceptions are observed before 1982 in the figure. In 1984, vibration causing the perception to 30% occupants or more almost did not occur even for buildings designed with β=-1.0.

The average tendency is shown in Fig.3, where the average number of days of perception for 10 years are shown with β as the abscissa. In this example, the variability of the response calculation is neglected. In buildings with β=-0.5, 30% occupants or more perceive wind responses in 4 days per year and in buildings with β=0.5, they perceive

Table 1. β categories for wind response

β value	human reactions
less than 0	complaints will occur
0 - 1	complaints may occur
1 - 2	perceptible but no complaints
greater than 2	not perceptible in majority

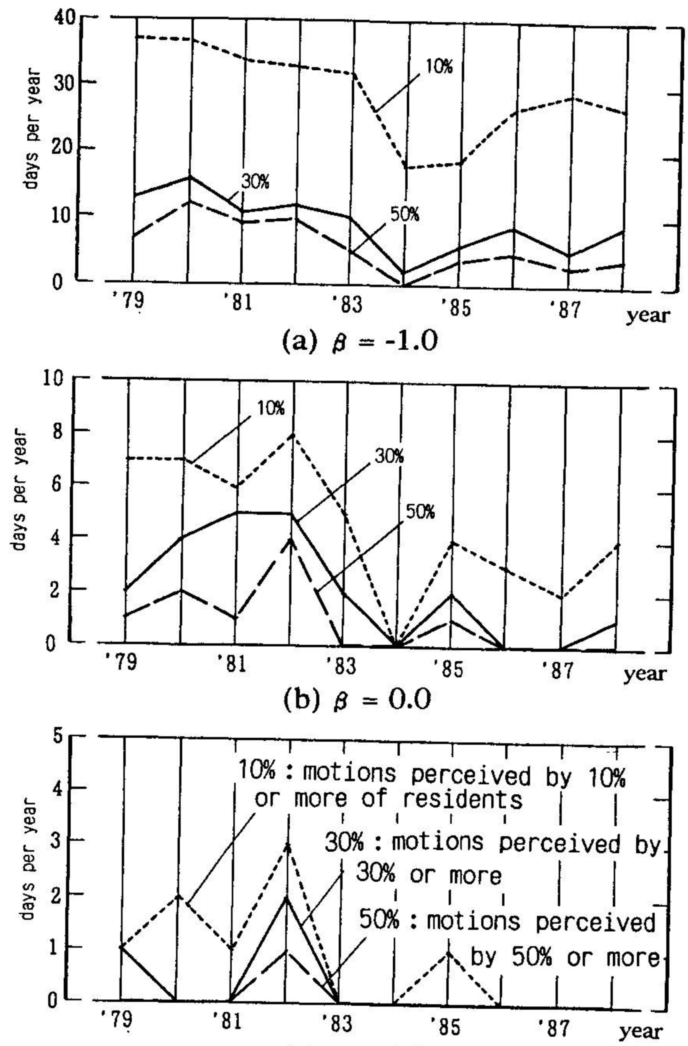


Fig.2. Annual change of number of days when occupants perceive motions due to wind

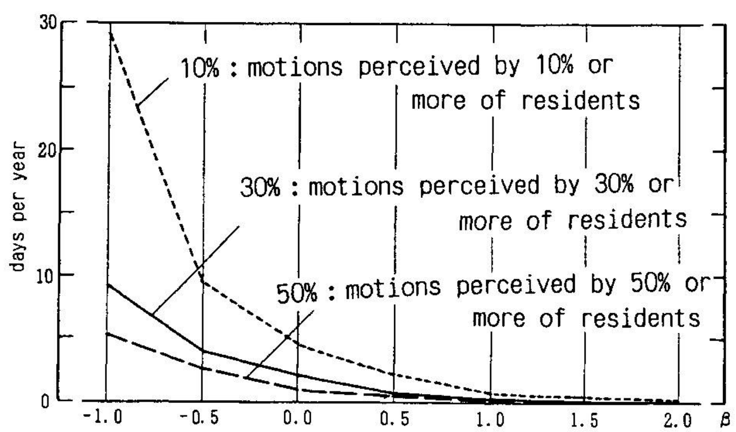


Fig.3. Average number of days for pereption of motions due to wind with β

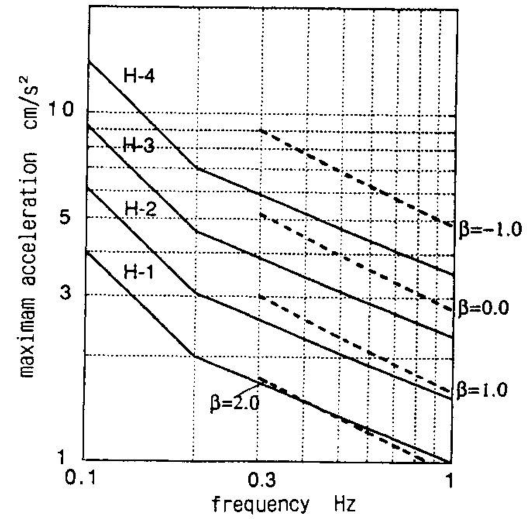


Fig.4. Comparison of perception criteria for wind response

motions in only one day per year as an average. Such numerical studies have to be followed in actural situations in order to reach a concensus on the target β_T among engineers, architects and occupants.

4. COMPARISONS WITH EXISTING GUIDELINE

Most exiting guidelines for the serviceability criteria of wind responses are deterministically specified to a wind speed corresponding to a return period. [3),4)] One of recent guidelines is shown in Fig. 4, when a design wind speed of one year retrun period obtained from daily maximum wind speed data is recommended to be used. The lowest level, i.e., H-1, is approximately consistent to ISO minimum perception limit .[5)]

For those criteria, the design format may be written as,

$$\hat{P} \geq \hat{A}_R \tag{5}$$

where $\hat{P}$ is the perception limit and $\hat{A}_R$ is the wind response amplitude due to the wind speed of R-year return period. When statistical data for the human perception of motion is utilized, $\hat{P}$ can be specified in a probabistic manner. For example, when P is modeled by the log-normal distribution with the mean of eq.(1) and the c.o.v. of 40%,

$$\hat{P} = \mu_p e^{-2 \zeta_p} = 0.685\, f_o^{-0.6} \tag{6}$$

also gives a limit close to H-1 in Fig.4.

Probabilistic serviceability limit state design format can be rewritten, based on the log-normal distribution model, from eq.(3) as ,

$$\mu_p \geq e^{\beta \sqrt{\varsigma^2{}_p + \varsigma^2{}_A}}\, \mu_A \tag{7}$$

In order to examine the relationship between probabilistic criteria in terms of β in eq.(7) and the deterministic criteria due to eq.(5), a parametric study was conducted. The c.o.v. of annumal maximum wind speed, V_{va},the uncertainty in the response prediction in terms of c.o.v., V_G, the c.o.v. of individual perception limits, V_p, and the retrun period R are chosen as variables. When the variability of the annual maximum wind speed and the uncertainty of the wind response prediction are given, Prob[A>P] can be calculated for the deterministic criteria with the design wind speed of return period R.

The c.o.v. of annual maximum wind load is assumed to be approximated by 2 V_{va}. Then the wind load for R-year return period, W_R, is obtained based on the Gumbel distribution for the annual maximum wind load as,

$$\widehat{W}_R = [\, 1 + \{-0.78 \cdot 2V_{V_a} \ln(-\ln(1-\frac{1}{R})) - 0.9\}V_{V_a}]\, \mu_{W_a} \tag{8}$$

where μ_{wa} is the mean of annual maximum wind load.

The peak acceleration response for R-year return period, $\hat{A}_R$, may be obtained by multiplying a coefficient C as,

$$\hat{A}_R = C\, \widehat{W}_R \tag{9}$$

Although C may not be a constant in a wide range of wind load, a linear relationship between $\hat{A}_R$ and $\widehat{W}_R$ is assumed for the simplification. Eq.(6) is also assumed, i.e., the deterministic perception criteria is given as the probability point which is specified as (the mean) -two times (standard deviation) for the log-normal distribution, namely,

$$\lambda_p - 2\,\zeta_p = \ln(\hat{A}_R) \tag{10}$$

When the c.o.v. is on the order between 0.2 and 0.5, the Gumbel distirbution can be approximated by a log-normal distribution. In particular when coefficient C is a random variable, the product of C and W_a can be well represented by a log-normal distribution.[6]

Then the perception probability is obtained by eq.(3) with $V_A^2 = (2V_{va})^2 + V_G^2$, $\mu_A = C\mu_{wa}$. The annual perception probability $\Phi(-\beta)$ is calculated and listed for V_{va}=0.1, 0.2 and 0.3; V_p=0.3, 0.4 and 0.5; V_G=0.0, 0.2 and 0.4 in Table 2 (a) for R=2(years) and (b) for R=5(years).

For a typical case of V_{va}=0.20, V_p=0.40 and V_G=0.20, the perception probability, P_p, values for R=2(years) and R=5(years) are 9% and 3% respectitvely. P_p for R=5 is not sensitive to the change of V_{va}, V_p and V_G in comparison with that for R=2. By utilizing the wind data in Tokyo, proposed criteria with various β which are shown by dotted lines, are compared with A.I.J. criteria introduced in Fig. 4. The peak factor commonly used in the

Table 2. Perception probability for the criteria specified for a return period wind[1)]

(a) 2-year retrurn period wind

(i) V_{va} = 0.10

V_p	V_G = 0.0	0.20	0.40
0.3	0.053	0.072	0.108
0.4	0.040	0.052	0.076
0.5	0.034	0.042	0.059

(ii) V_{va} = 0.20

V_p	0.0	0.20	0.40
0.3	0.110	0.132	0.140
0.4	0.077	0.085	0.101
0.5	0.059	0.065	0.078

(iii) V_{va} = 0.30

V_p	0.0	0.20	0.40
0.3	0.157	0.159	0.167
0.4	0.112	0.117	0.155
0.5	0.086	0.090	0.098

(b) 5-year return period wind

(i) V_{va} = 0.10

V_p	V_G = 0.0	0.20	0.40
0.3	0.018	0.030	0.059
0.4	0.016	0.024	0.043
0.5	0.016	0.021	0.034

(ii) V_{va} = 0.20

V_p	0.0	0.20	0.40
0.3	0.029	0.037	0.055
0.4	0.022	0.027	0.040
0.5	0.019	0.022	0.031

(iii) V_{va} = 0.30

V_p	0.0	0.20	0.40
0.3	0.040	0.044	0.055
0.4	0.029	0.032	0.040
0.5	0.023	0.025	0.032

random vabration theory, which is 3.74 for f_0=1, is used to calculate the maximum amplitude in A.I.J. criteria, while 2 is tentatively used in the proposed one as mentioned previously. β=2.0 corresponeds to the H-1 level and β=0.0 corresponds to a level between H-3 and H-4. Since V_{va} and V_G vary depending on site and design situation, perception probability consistent criteria seem to be preferable to a conventional criteria with a specified return period wind speed. The flexibility in the specification of building performance is described by the second moment reliability intex β. A more consistent β value should be achieved through many opportunities of use of the serviceability limit state design with proposed probabilistic criteria, although it may take time to make such an index as a common measure for wind response performance in the society.

5.CONCLUSION

Probabilistic design criteria with the second moment reliability index are proposed based on experimental data on the human perception limits to low-frequency horizontal motions. A comparison with one of recent guidelines with conventional criteria was made by demonstrating numerical examples. The flexibility in the specification and the rationality for allowing the variability of both wind loads and individual perceptions are stressed.

ACKNOWLEDGEMENT

This study is a part of the research conducted in the committee on "Habitability performance of highrise residential buildings in strong winds" supported by Housing and Urban Development Corporation and Technology Research Center for National Land Development.

REFERENCES

1. KANDA, J., TAMURA, Y. and FUJII, K., Relationship between Probabilistic Criteria and Deterministic Criteria for Human Perception to Wind Induced Motions, Proc. Serviceability of Steel amd Composite Structures, Pardubice, 1990, pp.61-66.
2. KANDA, J., TAMURA, Y. and FUJII, K., Probabilistic Perception Limits of Low-Frequency Horizontal Motions, Proc. Serviceability of Steel and Composite Structures, Pardubice, 1990, pp.67-72.
3. Koten. H. van., The Comparison of Measured and Calculated Amplitudes of Some Buildings and Determination of the Damping Effects of the Buildings, Proc. Wind Intern. Conf. Wind Effects Build. Stru., Tokyo, 1971, pp.825-839.
4. Architectural Institute of Japan, Guidelines for the Evaluation of Habitability of Building Vibration, April 1991.(in Japanese)
5. I.S.O., Guideline for the Evaluation of the Response of Occupants of Fixed Structures, Especially Buildings and Off-Shore Structures to Low Frequency Horizontal Motion (0.063 to 1 Hz), ISO 6897, lst ed., 1984.
6. SUNOHARA, H. and KANDA, J., Probability Distribution of Yearly Maximum Wind Load, Rep. Ann. Meeting of Archit. Inst. Japan. Str. I., Oct . 1985, pp.675-676. (in Japanese)