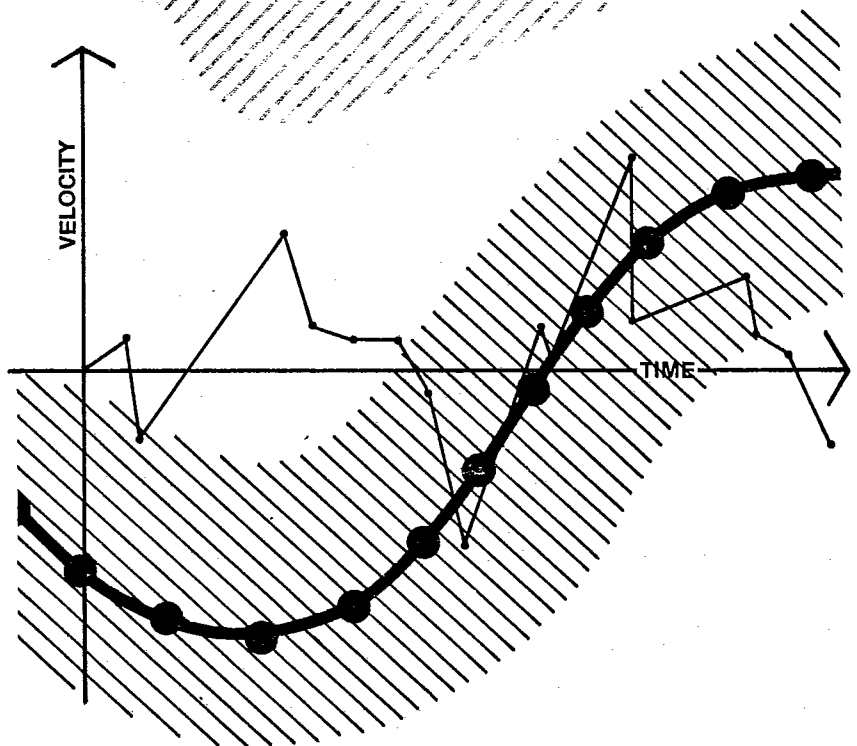


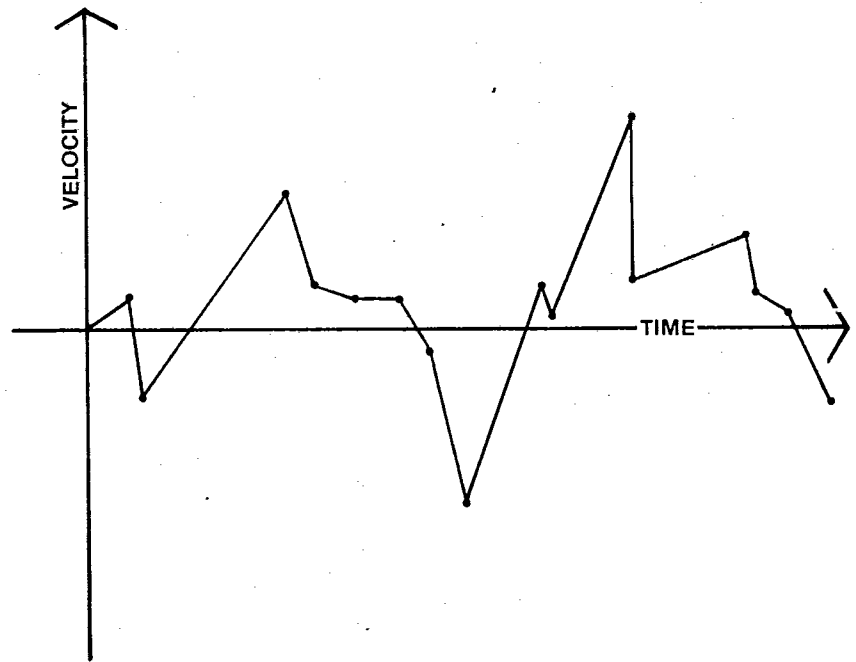
Vibrations of Concrete Structures



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PREFACE

The present volume includes the papers presented at the symposium on Vibrations of Concrete Structures held on October 20, 1977 at the Fall Convention of the American Concrete Institute in New Orleans, Louisiana, as well as others which could not be presented because of time limitations.

It is to be noted that this volume deals with dynamic deflections of concrete structures and thus, it forms the second part of the previous publication, SP-43, Deflections of Concrete Structures (1974) under the sponsorship of the same committee.

The first three papers were prepared at the request of committee 435 in order to present an up-to-date, state of the art in the area of vibrations of structures and, background and the origin of this symposium. The next three papers were also requested by the committee from individuals well known in related field on specific topic to make this a balanced and complete volume on the subject matter.

The remaining papers cover a range of vibrations related subjects as experienced by many researchers, academicians and practicing engineers. The last item in the volume is the bibliography of references that has been collected by the members of committee 435. It is the hope of the committee that the volume will help substantially to many, who might face a problem with Vibrations of Concrete Structures.

The timely publication of this volume was made possible by the efforts of all the authors, who completed and prepared their manuscripts within a tight schedule, the members of ACI 435, who reviewed all the manuscripts according to the ACI publications criteria, and the ACI staff.

Gajanan M. Sabnis, Chairman
Symposium Subcommittee and
former Chairman of ACI 435

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SP 60-1

Introduction and Background: Vibrations of Concrete Structures

By Gajanan M. Sabnis

Synopsis: Various aspects of vibrations of concrete structures are presented to set the stage for the rest of the symposium. These include, complexity of vibrations, structural problem, human response, approach for their evaluation and elimination and code provisions. It is hoped that the information provided in this paper will stimulate the thinking of designer concerning the potential problem that he might be faced with.

Keywords: building codes; dynamic characteristics; evaluation; human factors engineering; prestressed concrete; reinforced concrete; structures; vibrations.

Dr. Gajanan M. Sabnis, F ACI is an active member of ACI, and has been until recently chairman of ACI committee 435, Deflections of Concrete Structures and also is a member of ACI Committees 116, 444, 549. He is also active locally in ACI activities and presently is Director of the National Capital Chapter, of which he was a co-founder and past President. Dr. Sabnis obtained his Ph.D. from Cornell University and is registered engineer in the States of California and Maryland. He is currently associate professor in Civil Engineering at Howard University and is editor of Handbook of Composite Construction Engineering to be published by Van Nostrand Reinhold.

INTRODUCTION

When this symposium was planned, the ACI committee 435 had just begun to think in terms of the effect of dynamic action of forces on concrete structures from the deflections point of view. For many years, engineers have faced this problem and looked for assistance to those who may know a little bit more about it and possibly the solution. The committee started looking seriously into this topic and especially having just completed the task of 'state-of-the-art' on static deflections(1), came up with the excellent idea of doing likewise in the case of dynamic deflections(2).

It is the express purpose of this paper to touch upon various background items in order to set a stage for the symposium.

The remaining papers of this symposium deal with specific problems and occasionally with recommendations as to how a solution might be reached. In some instances, research background is presented which should be helpful to designer.

Various aspects of vibrations in concrete structures are:

1. existence and complexity of vibration
2. structural problem of vibration
3. human response to vibration
4. approach for evaluation and remedies of the problem
5. some code provisions

EXISTENCE AND COMPLEXITY

Vibrations in a structure occur due to a multitude of causes. They are commonly caused by walking (as

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inside a building) or by wind or other loading (external to the building). Other forms of loading causing vibrations are considered in several of the papers in this symposium that follow. They include blast loading (3,4), seismic vibrations (5), construction loading (6) and so on. The main reason for importance of vibration in a structure is attributed to the concern for the serviceability due to use of higher strength materials and the use of strength theories which in turn results lower thickness of floors and hence may accentuate the deflections due to vibratory loading.

Floor vibrations is an oscillatory motion of the floor about a position of rest. When a panel vibrates it may create vibration of opposite nature in the adjacent panel. This may cause discomfort to the occupants of the adjacent panel. The original vibration and the translated vibration will cease with the passage of time as shown in Fig. 1 (6) depending on the damping characteristics of the floor. The higher the stiffness, and the higher the damping, the faster will be this decay and the less discomfort will be experienced by the occupants. This figure also shows that higher frequency modes can be neglected since they alternate quickly and do not cause serious discomfort. The effect of vibration is further complicated by the fact that different people will react differently, depending on their ability of perception and on many other factors.

THE STRUCTURAL PROBLEM

The term "structural problem" refers to that which the structure itself will be faced with in the event of vibrations. The problem has been recognized for many years. In 1828, Tredgold wrote (9):

"Girders should always, for long bearings, be made as deep as they can be got; an inch or two taken from the height of a room is of little consequences compared with a ceiling disfigured with cracks, besides the inconvenience of not being able to move on the floor without shaking everything in the room."

The above quotation clearly signifies the problems, which have been recognized by the codes with our simplified provisions for depth/span ratio and for deflection limits under equivalent static loading. It is now recognized, however, that the designer must calculate certain dynamic characteristics, such as frequency and damping of the structure in order to prevent or minimize potential problems.



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Frequency (f) of a simply supported, one way system (9) is given by:

$$f = 31 \sqrt{\frac{E}{\omega L^4}} \quad (1)$$

in which the terms are

E = Modulus of Elasticity
I = Moment of Inertia
 ω = load per unit length of span
L = span

(ALL units are in pounds and inches)

The values for other support conditions can be found using different coefficients as shown by Blume and Honda (5). Proper considerations must be given to account for other attached members in the above expression.

Fig. 2 shows how the natural frequency of reinforced concrete can be determined. The details were developed using following frequency expressions

$$f = \frac{\pi}{2} \left\{ \frac{1}{\ell_x^2} + \frac{1}{\ell_y^2} \right\} \sqrt{\frac{D}{\rho t}} \quad (2)$$

$$\text{where, } D = Et^4 / 12(1 - \nu^2) \quad (3)$$

in which the terms have the usual meaning. Both the simply supported and built-in edges are considered. It is recommended (10) that the thickness should be adequate to have a frequency value of higher than 15 Hz.

Damping in a floor system must be estimated. A range of values for damping is suggested (9). This range may vary between 2 per cent for bare concrete floors to 10 per cent for finished floors with concrete deck and partitions in place.

Damping will absorb the vibrations in a certain frequency range and will reduce resonance and in turn any potential damage to the structure. The relation between damping expressed in percent of critical damping and decay of free vibration is shown in Fig. 3.

HUMAN RESPONSE

There are basically three kinds of human exposure to vibration, namely:

- (a) Vibrations transmitted simultaneously to the whole body surface or substantial parts of it. e.g. when

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high intensity sound in air or water excites vibrations of the body.

- (b) Vibrations transmitted to the body as a whole through the supporting surface, namely, the feet of a standing man, the bottom of a seated man or the supporting area of a reclining man. This kind of vibration is usual in vehicles, in vibrating buildings and in the vicinity of working machinery.
- (c) Vibrations applied to particular parts of the body such as the head or limbs; for example, by vibrating handles, pedals, or head-rests, or by the wide variety of powered tools and appliances held in the hand.

It is also possible to recognize the condition in which an indirect vibration nuisance can be caused by the vibration of external objects in the visual field (for example, an instrument panel).

Human response in these conditions will depend on many factors as listed below:

- 1. Type of excitation (e.g. impulsive shock, machinery vibration, etc.)
- 2. Type of property (e.g. residential, office, hospital, etc.)
- 3. Time of day
- 4. Limit of acceptance

The first two characterize the structural problem as was mentioned in the last section. The last two depend on an individual's position, when subjected to vibration and the effect will accordingly change. Examples are shown by three positions as shown in Figure 4 to determine such a response (11). Since the effect will differ, suitable weighting factors are recommended to set up the proper criteria for the type of occupancy.

APPROACH TO EVALUATION AND REMEDY

For a given structure and occupancy the dynamic characteristics may be evaluated as mentioned before and suitable structural deficiency made up. On the other hand, other remedial measures for continuous vibrations (9) can be:

- (a) Relocation of vibrating source
- (b) Removal of source
- (c) Vibration isolation

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Sources, such as machinery or potentially usable areas for dancing or gathering, should be isolated from sleeping quarters or office areas. Structures should be designed (or altered) to avoid resonance by making the floor frequency of the sensitive occupancy different from that of the floor directly vibrated. Equipment should be balanced to reduce eccentric forces or connected to the structure with large mass to increase inertial resistance to movement. Often, vibration sources can be founded on isolated pads or soft springs to reduce transmission of vibrating to the structure. It must be recognized that isolation will only succeed if it is less than the forcing frequency. e.g. if it is $1/3$ of the forcing frequency, the transmitted frequency is reduced approximately 80 per cent. On the other hand, if the two are about the same, the transmitted vibration is greatly enhanced due to resonance.

Structural properties mentioned before can be used to avoid resonance and to provide suitable damping to reduce the duration of vibrations.

CODE CONSIDERATIONS

As mentioned before the problem of vibrations has drawn more attention of professional engineers in the last decade due to the use of higher strength and lighter materials leading to economical but shallower structures. It is hoped that based on ISO* standards, codes will be able to recommend provisions for vibration standards. These criteria should consider:

- (a) human response to vibrations
- (b) limitation of structural stiffness

ISO specifies exposure limits for vibration transmitted to human body from solid surfaces in the range of 1 to 80 Hz. These standards should be specified in terms of vibration frequency, acceleration magnitude, exposure time and the direction of vibration relative to the torso.

The basic rating (Hz) is given for the most stringent conditions and proper weighting factors applied for other conditions depending on the type of structure and occupancy.

The stiffness of a floor should be such that maximum deflection shall not exceed certain limits. According to ISO (11), this limit with a standard 150 lb. static line load distributed over a floor width is taken as:

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$$\Delta_v \leq \frac{2}{3f} \quad (4)$$

in which Δ_v = maximum deflection, in inches

f = lowest natural frequency of the structure in Hz.

Canadian Standards Association (CSA) (12) recommends that proper corrective measure should be taken to reduce the annoying vibrations, if they exist. The effective way is to increase damping by adding partitions or damper posts in the floor below. If these methods are not suitable, special devices to absorb vibrations or damping materials should be incorporated into the floor system. Even a simple measure of using a rug is effective in reducing walking impact as well as in cushioning the sway of china cabinets.

DIN 4150 (13) takes slightly different approach and provides with equations to calculate maximum stress in a structure subjected vibration velocities, which in turn are related to human exposure. Gasch and Klippel (4) have discussed this in detail. Fig. 5 shows relationship between the various quantities as contained in the German provisions.

Although currently no provisions exist in the ACI code, it is felt that based on what is presented in this symposium and the material available to the profession, ACI Committee 435 will eventually make recommendations to ACI Committee 318 for inclusion in future building codes.

*International Standards Organization

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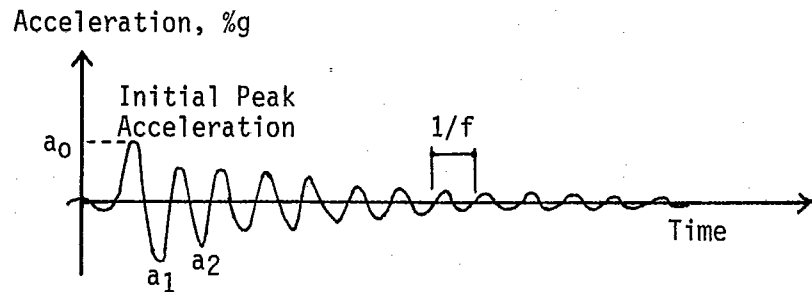


Figure 1 Typical Transient Vibration from Heel Drop
(High Frequencies Filtered out) (From Ref 12)

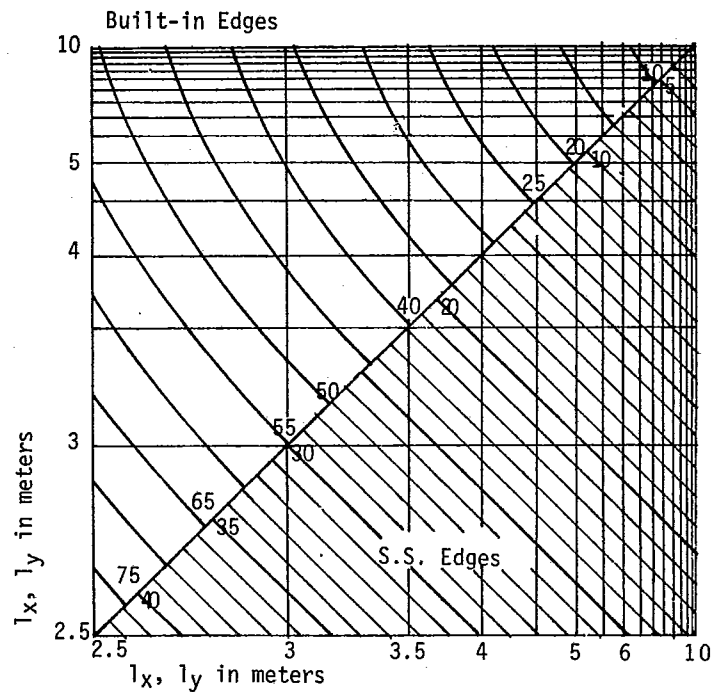


Figure 2 Relation between Natural Frequencies and Spans
for Reinforced Concrete Slabs (From Ref. 10)
($t = 10$ cm)

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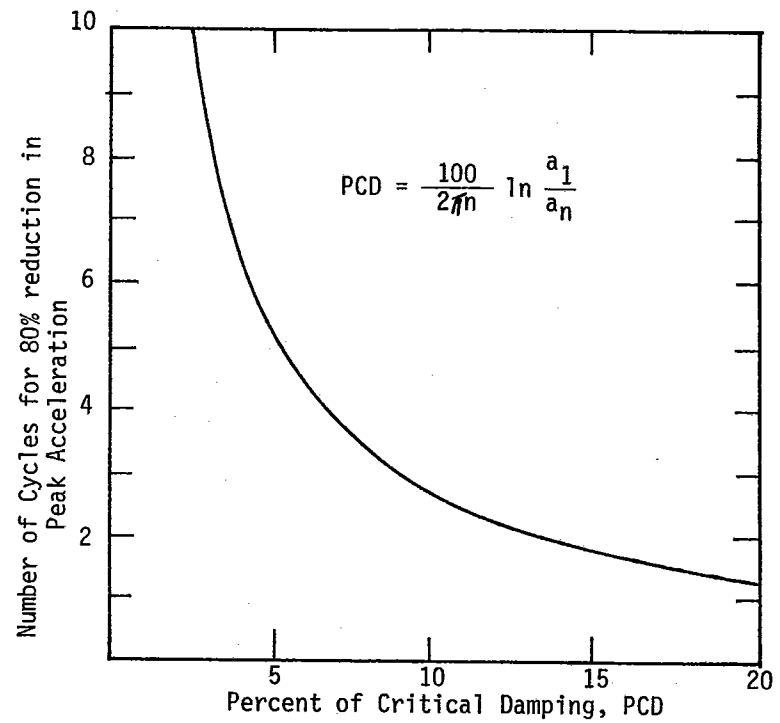
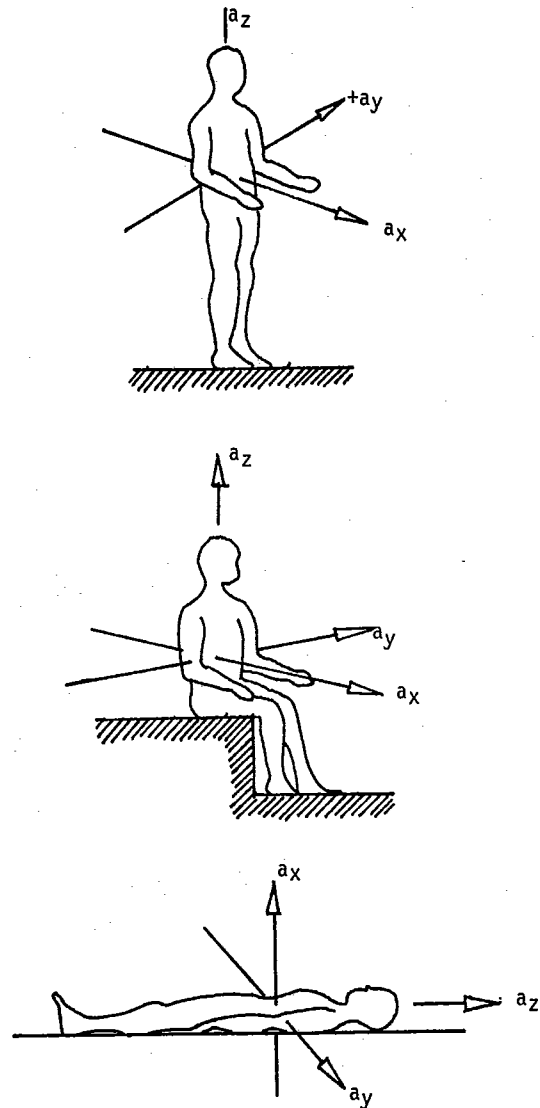


Figure 3 Relation between Damping and Decay (From Ref.12)

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a_x , a_y , a_z = Acceleration in the Directions of the
 x, y and z axes
 x- axis = back to chest
 y- axis = right to left side
 z- axis = foot to head

Figure 4 Coordinate System for Mechanical Vibrations Influencing Humans (From Ref. 11)

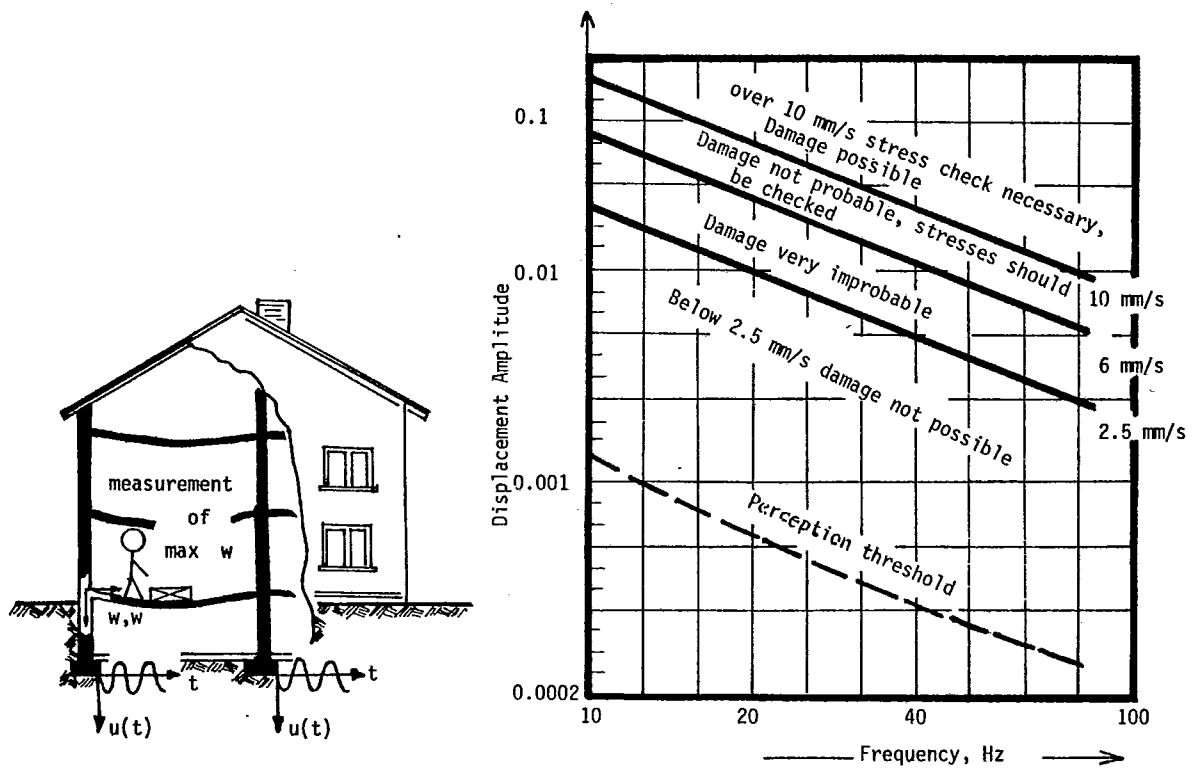


Figure 5 Evaluation of the Effect of Steady-state Floor Vibrations by Measurement of the Vibration Displacement Amplitude and Frequency or by Measurement of the Vibration Velocity (Based on German Standard for Vibration Protection) (From Ref. 4)

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Deflection Requirements— History and Background Related to Vibrations

By J. Warwaruk

Synopsis:

This paper reviews historical developments in deflection requirements and current deflection requirements as they relate to vibrations in structural members and buildings. A brief review of the building process itself is also presented. For members the deflection requirements relate to effect of both human and other live loads. In the case of buildings the vibrations induced and limitations cited relate principally to wind loadings.

Deflection limitations currently in use as a measure of serviceability for members and buildings are cited and discussed as to their applicability and short coming.

Keywords: criteria; deflection; human factors engineering; reviews; serviceability; structural members; structures; vibration; wind pressure.

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INTRODUCTION AND OBJECTIVE

The problem of deflection requirements for vibrations in structures has been recognized and primitivity considered for a long time but only recently has been brought into sharp focus due to improved and new materials, new design techniques, and a larger volume of tall building structures. Although this problem is yet not fully solved, considerable progress relating to the study of effects of vibrations on buildings and humans has been made within the last two decades.

The objective of this paper is to review pertinent developments in deflection requirements as they relate to structural vibrations. In considering historical developments included is a review of the building process together with a review of deflection serviceability requirements including developments in wind loading. Current deflection serviceability requirements are discussed together with new developments especially as they relate to human response.

Regarding serviceability it is defined in the usual sense. Generally a member or structure is considered serviceable if its performance under specified conditions meets specified limiting conditions.

Vibrations in structures may result from various sources. Steady state vibrations usually result from rotating machinery and will not be discussed in detail since they can be isolated easily. Transient vibrations result from a variety of forcing functions, the most significant being earthquake and wind. Only wind is considered in this report. Earthquake loading, although a necessary consideration in building design but because it is most important at the collapse limit state, is not discussed. The effects of minor earth tremors at service load levels would, in the first instance, be similar to the effects of wind.

REVIEW OF THE BUILDING PROCESS

The earliest form of structure was the post and lintel system of wood or masonry either brick or stone, depending on local availability of materials. By the beginning of the nineteenth century a great deal of experience, skill and dexterity was achieved in the use of walls, columns, lintels, vaults, arches, and domes for buildings.

In America from the earliest times lumber was the most used building material as compared to stone and brick, until the latter part

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of the nineteenth century. Iron was introduced into American building construction about 1820 but significant changes in building construction took place only in the latter half of the nineteenth century after the steel manufacturing process by Bessemer was developed. Use of steel in buildings marked a very significant milestone in the history of building construction as it permitted not only the rapid construction of buildings of sufficient strength and rigidity but also to proceed with construction of the frame more rapidly than previously with bearing wall construction, and to follow with curtain walls at an independent pace.

The first tall building utilizing steel in its framework was the ten story Home Insurance Building built in 1885 in Chicago (1). This was followed by the 16 story Monadnock Building, 1891, which had portal bracing for resisting wind stresses and exterior load bearing masonry walls. At the same time tall buildings were being erected in New York but the construction was mainly steel and masonry bearing wall, an example of which was the Tower Building built in 1889. The fifteen story Havemeyer Building, 1892, had the interior and wind loads carried by an iron frame with double diagonal bracing in the outer bays and masonry walls carried peripheral floor loads. The braced and riveted steel skeleton established itself in New York in 1895 with the construction of the American Surety Building, a twenty story building without masonry walls.

The beginnings of reinforced concrete in buildings go back to about the middle of the nineteenth century where it was first used in a reinforced concrete floor slab by Wilkinson in England. The initial early experiences with reinforced concrete in structures were only moderately successful in that reinforced concrete was designed and used in structures in a manner similar to that of timber and steel with no regard given either to the monolithic nature of concrete or to the potential of continuous elements in structures reducing the bending moments. Among the early notable structures using reinforced concrete in America were those of Ward who built a reinforced concrete house in 1872 (1), and Ramsome who erected several buildings in California including the museum built on the campus of Stanford University in 1892. In reinforced concrete structures the manner of manufacture was either using load bearing walls similar to that of stone, brick and masonry, or, frame structures as that of steel. In the former method, exterior walls were designed to be of sufficient strength to carry the loads of beams, girders, floors and roofs which rest upon walls, and on in the interior, either walls or columns served as supporting members. In the latter method, floors and roofs rested directly on exterior and interior columns, and curtain walls of brick, masonry, or stone were used to enclose the structure. With continued experience the frame was used for most reinforced concrete structures of any size.

In summary, during the nineteenth century more significant changes and developments occurred in methods of building construction than in any other given period of human history. Most notable was the use of steel and concrete for buildings of extended heights, made possible by the introduction of the skeletal steel frame and the reinforced concrete frame, and in both, the use of curtain walls for exterior enclosures, as contrasted to the centuries old technique of

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using load bearing walls which resulted generally in structures of low overall height. Although buildings during the nineteenth century continued to be built in traditional ways using brick and timber new roles emerged for materials formerly used in load bearing applications. Brick, stone and masonry in buildings of any permanence were being used for curtain walls and in many instances, especially stone, used to embellish building interiors.

Developments in techniques of building construction in the twentieth century have been more dramatic than those for the nineteenth century. In the period up to World War II the primary materials for building structures continued to be steel, concrete, and timber. For each, improvements occurred in design techniques, in manufacturing techniques, and in joining or fastening techniques.

Steel continued to be used extensively for structures and riveting constituted the main method of forming connections. Bolts for fastening were used but not extensively since it was considered that vibrations from machinery and wind may tend to loosen them. Welded connections were introduced and use of welding for fabrication of structures developed significantly in the decade from 1930 to 1940. Construction of tall buildings using steel frameworks continued especially in New York and Chicago and most notable of these is the Empire State Building (2) completed in 1931, with an overall height of 1239 feet.

Reinforced concrete advances in the period before World War II were numerous and significant. Not only were there improvements in cement quality yielding higher concrete strengths and improvements in the quality and types of reinforcement, but also innovative and new structural forms emerged. A notable development was the flat slab structure developed by Turner of Minneapolis in 1906. Many other systems of reinforcement were patented in the early 1900's and these are described in detail (3) together with numerous load tests which were made on these slabs. Other significant developments included the extensive column investigation of the American Concrete Institute (4) which resulted in a factored strength design for columns rather than working stress design, and, investigations of beam and footing members. In terms of design, working stress design techniques were used throughout this period with increased levels of working stress both in concrete and reinforcement at the end of the period as compared to those at the start, reflecting the improvements, over this period, in material properties.

Use of timber and wood in building construction has continued in the twentieth century but at a reduced rate compared to that of steel and concrete and with an increased emphasis in applications towards interior and exterior building finishes. Significant developments included classification of lumber into grades, connectors such as split rings and shear claw plates for tension members, and, the lamination of smaller stocks into a larger member.

Developments in building technology since the end of World War II have been rapid, many and varied. With respect to structure itself, steel and concrete continued to be the primary materials used in large structures, and, wood and masonry generally used in small buildings. This division reflected the capability of steel and concrete to carry the heavy loads of large buildings economically.



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In this period there are many structural developments utilizing steel, and by way of summary these include continuous trusses, space trusses, Vierendeel trusses, rigid hollow boxes, cable suspended roof systems, metal space shells, and geodesic domes. Fabrication techniques now include welding to a large degree since considerable improvements in welding materials and procedures occurred, and also, it is feasible now to inspect welds almost anywhere using dye penetration techniques, radiographic or ultrasonic techniques. Riveting lost its former prominence and field connections using high strength bolts have been common since about the middle 1950's. In design and construction greater advantage is now taken of moment connections and continuity. An example of a structure typifying welded steel framing is the Chicago Civic Center (1). Also, more recently several buildings have employed rigid frame bents or trusses on the exterior for lateral load bracing, examples include the IBM Building in Pittsburgh, the Alcoa Building in San Francisco, and the Hancock Building in Chicago.

Steel is achieving prominence in building floor systems and is now used in lightly loaded floors as thin walled cold rolled joist sections in many cases simply a replacement for wood joists, and, extending to the combined use of open web joists together with either reinforced concrete, gypsum plank or corrugated steel and concrete floor slabs for heavier loads.

The history of developments in reinforced concrete in post-World War II includes both material improvements and structural assembly improvements. Cement quality improvements now permit concrete strengths in excess of 5000 to 6000 psi. Reinforcement developments include not only higher strengths having yield levels in excess of 60 to 80 ksi but also defined surface deformations improving greatly the interaction of reinforcement and concrete. In this period prestressed concrete emerged and prestressed steels having strengths ranging from 250 to 300 ksi were developed. It permits the use of shallower members for a given loading and span than that possible with reinforced concrete. In its applications to buildings prestressed concrete has gained considerable advantage even over reinforced concrete in that the elements may be fabricated in a factory away from a building site, transported and then "tied" together at the site.

Slab construction for floor systems carrying heavy loads continued to be built using column capitals and drop panels and was extended in application to the very simple flat plate by eliminating the column capitals and drop panels for applications having lightly loaded floors. Vertical members of reinforced concrete structures experienced developments beginning with the usual columns cast monolithically with slabs, to load bearing screen walls where the exterior beams and columns constituted a continuous vertical framework, then to thin wall members cast monolithically with floor slabs to form a box system. In all cases the continuity between vertical and horizontal members provided some restraint against lateral loadings but with continued increases in heights of buildings this became insufficient and the vertical shear wall emerged as a structural element. Typifying the combination of vertical shear walls on the interior and exterior frames in a structures is the Brunswick Building in Chicago (1).

Walls and partitions have experienced a most rapid change

since World War II. Traditionally, in addition to glass, brick or stone masonry either of the bearing or curtain wall type constituted walls in large buildings while wood and brick or stucco were used in small buildings. Interior partitions were normally of wood and plaster.

Developments in curtain wall construction since World War II have included extensive use of plastics or light gage metals such as aluminum and steel together with insulating materials. In many framed structures glass is used extensively as the material constituting the wall. Curtain walls using wood consist either of a waterproof plywood or a sandwich panel in which plywood is one of the components, or siding. In the case of frame structures using panel type curtain walls the practice currently involves the provision of a secondary framework system made either of wood or metal which holds panels in place. Such a secondary framing scheme usually consists of vertical members or mullions spanning between floor slabs and horizontal members such as headers, sills and transoms between the verticals. These exterior walls carry not only their own weight between supports but they must also be capable of resisting wind forces and of transmitting them to the primary structure. Materials used in interior partitions at the present time include wood or light steel studs faced with either plaster wall board or plywood or plastic and any other material which meets functional and architectural requirements.

HISTORICAL DEVELOPMENT - DEFLECTION REQUIREMENTS

Members--The historical development of serviceability requirements for structural members parallels the development of the building process. Since the beginning nineteenth century a deflection limitation has been in existence as a requirement, additional to strength, in proportioning beam members. It was first used in limiting the total deflection of wood beams which supported a plastered ceiling below. Russell (5) presented a detailed account of the probable origin of this deflection limitation and he credited the first recorded mention of such a limitation to Thomas Tredgold, an English civil engineer who suggested a maximum deflection of $1/40$ of an in. per ft. of span or $1/480$, in his book "Elementary Principles of Carpentry" published about 1820. It is significant to note that Tredgold recognized the importance of stiffness of a beam not only for preventing "a ceiling disfigured with cracks", but also, "the inconvenience of not being able to move over the floor without shaking everything in the room".

In tracing subsequent developments regarding deflection limitations Russell (5) indicated that a great deal of progress occurred between 1820 and 1872 when Trautwine first published "The Civil Engineers Pocketbook". During this period more experience was gained not only in the performance of beams using Tredgold's basic design limitation but also in the extension of this limitation to beams of other materials. Trautwine presented deflection limitations of Tredgold but suggested that values of limiting deflection could be increased to $1/30$ of an in. per ft. of span and indeed, such a value was beginning to be used in America. By 1885 Kidder (6) indicated that $1/30$ in. per ft. of span or $1/360$ of the span is not too much to

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allow for the deflection of floor beams as a floor is seldom subjected to its full estimated load and then only for a short time. In this period statutory loadings were being incorporated into building codes such as those in New York in 1885 which required a superimposed load of 75 psf in all buildings and 120 psf for public assembly buildings. Kidder suggested that in dwelling houses, etc., it is not necessary to allow more than 40 psf for superimposed loads. In 1906 Mitchell (7) indicated that beams directly supporting plastering should not deflect more than $1/480$ of their span otherwise the plastering will tend to crack; in other cases $1/360$ of span should be the maximum deflection permissible. In 1911 Trautwine (8) stated that "to prevent cracking of the plaster of ceilings it is usual to limit the deflection of beams to span/360 or $1/30$ in. per ft. of span".

Kidder-Parker (9) in 1931 indicated that a stiffness factor of a beam δ/L , based on experience should be $1/360$ where beams support a plastered ceiling. It was further suggested that because the deflection due to dead loads except for plaster itself had already occurred, a correct statement of deflection limitation to avoid plaster cracking is: "The deflection due to live load must not exceed $1/360$ of the span in inches." But, for first class work it was considered desirable by Kidder-Parker to limit the deflection to $1/360$ of the span where the deflection was that due to dead and live load. This reflects a further liberalization of the deflection requirement as a result of continued experience with the performance of building components. It is also of interest to note that Kidder-Parker suggested, in the case of timber beams, the limitation of vertical deflection to $1/360$ of the span should correspond to the use of full dead plus live load. They recognized further for wood beams the use of reduced modulus of elasticity in deflection calculations to account for continued increases in vertical deflections with time for no increases in load.

Gay and Parker (10) in the early 1930's presented a deflection limitation of $1/360$ of the span when beams carry a plastered ceiling or any other easily disrupted material. Although the corresponding loading for calculating deflection was not explicitly indicated, from the examples provided the full dead load and live load was used in these calculations. Live loadings used in building calculations were those recommended by the U.S. Dept. of Commerce.

Shedd (11) in 1934 also suggested it was usual to specify that beams supporting a ceiling be limited in their total or perhaps in their live load deflection and a common limit developed from experience and general observation is $1/360$ of the span. The reasons given for limiting live load deflection only is that the plaster is put on after practically all the dead load is in place and therefore only the further deflection due to live load needs to be considered. But Shedd stated further: "The author has not as yet found any positive evidence to support either position, other than the dogmatic statement of various authorities. It seems best however to limit the total deflection; limiting live load deflection only will result in a rather flexible structure in many cases." Shedd suggested a limiting ratio of depth to beam to span of the order of $1/25$ for buildings.

Hoole and Kinne (12) in the early 1940's indicated span depth limitations ranging from 25 to 30 for steel beams and of the order of 10 to 15 for wood beams. These calculations were based on a limiting



deflection of span/300 and particular values for working stresses and moduli of elasticity. At about the same time Grinter (13) suggested that deflection limitations be imposed on beams for two reasons: First "there is the problem of cracking plaster which can unquestionably be avoided if the calculated deflection is less than 1/360 times the span," and secondly beams of different size are often "bricked in" together and must deflect together and have identical calculated deflections. In the examples presented the deflections calculated are those corresponding to dead load plus live load.

For reinforced concrete little information was found in regard to limiting deflections for beams or even procedures for calculation of deflections prior to World War II. This most likely reflects no serious difficulties with structures of reinforced concrete. The monolithic nature of beam and slab systems, the low working stresses, the beneficial effects of torsional stresses especially in slabs, the neglect of tensile stresses, all contributed to a deflection which would be smaller than that calculated. Deflection limitations for reinforced concrete structural components were first presented in the 1963 ACI Building Code (14) except for flat slabs. Because the early structural uses of concrete were patterned after that of wood and steel, designs yielded members having sufficient stiffness so that problems with plaster cracking and vibrations were not common. In the early 1900's new and innovative ways of using concrete in structures were evolved, notably the flat slab system for floors and roofs. Since it was difficult to relate to theoretical designs the capacities of floor systems of that period in many buildings were "proofed" by load tests and in numerous cases only small deflections were observed for test loads as high as twice the design live loads. Such favorable performance removed the need for much further refinement in deflection limitations. Table 1 presents a summary of limitations for slabs extending from the early 1900's to the present.

Structures--The historical development of serviceability requirements regarding lateral deflection or drift for building structures is intimately linked with the development of wind loading requirements. From the earliest history of building construction the effect of wind on buildings has been recognized but only recently has considerable concern been given to a realistic assessment of wind effects.

Typical of the manner in which wind effects were considered at the turn of the century is that of Mitchell (7). Wind velocities and pressures were quantified relative to a qualitative description of the wind. Once the wind velocity was known the pressure was derived from an empirical formula such as $1/200$ times velocity squared equals pressure. Examples of wind velocity gradations cited are: slight breeze (4), brisk wind (25), high wind (40), violent storm (60) and violent hurricane (100), where the numbers in parentheses are suggested velocities in miles per hour.

The manner of building construction of the late 1800's and extending to a degree even today employing bearing walls and yielding heights of buildings of no greater than 4 to 6 stories with rather thick walls resulted generally in the wall being able to resist wind forces successfully so that no special precautions for wind were

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necessary. Even as late as the 1940's codes permitted buildings of certain heights without any provision for wind pressure; for example, a height of from one and one half to three times the least horizontal dimension was permitted before requiring wind pressure to be considered (12).

For skeleton frame buildings of either the mill type or high rise the inplane stiffness of the wall is normally not considered in assisting the frame to resist wind forces and the frame itself must be made sufficiently rigid to resist lateral forces of the wind. In the early history of skeleton frame buildings (15) special bracing of the truss type, portal type and rigid connections between beams, columns and girders was used to provide lateral stiffness. Regardless of the means of providing lateral stiffness in skeleton frame buildings concern was taken to guard against undue deflections of members and also vibrations which arose from such lateral loads. Spurr (15) indicated "with a correct choice of maximum wind load it is necessary only to limit the total deflection to a proper amount in order to keep the vibration within proper limits", and suggested a maximum total frame deflection of 0.001 of the building height or about 1/8 in. per story. The Bank of Manhattan Building described by Spurr (16) has wind bracing sufficient to provide a frame rigidity equivalent to a lateral movement of 0.001 of its height when subjected to an average wind pressure of 15 psf. Spurr (15) additionally recognized that for more slender towers a maximum frame deflection of 0.002 of the building height should be used. In a progress report (17) the ASCE Subcommittee dealing with wind bracing in tall steel buildings recommended that a deflection index, defined as a measure of stiffness and evaluated as a ratio of the maximum deflection at the top of a frame to the height under maximum wind load assuming the frame to carry the entire lateral force, of 0.002 is representative of satisfactory behavior in the matter of deflection and vibration. This subcommittee recognized that such an index would not be an actual deflection due to the inertia and the damping effects of walls, partitions and floors but some multiple of it and in this way could serve as a convenient standard by which to "judge the probable stiffness of a building and its probable freedom from disturbing vibrations". They stated further that "only experience will reveal the relation between computed maximum deflection and comfortable occupancy". Gay and Parker (10) indicated that "such vibration or tendency to sway when exceeding certain values is uncomfortable for the occupants and has a most unfavourable effect upon office rentals. To render a building safe against collapse from wind pressure is not difficult, but to arrive at exact allowable deflections, correctly chosen with regard to the human nervous system, requires more experiments and tests of actual horizontal movements in buildings of various types and proportions than yet have been made." In discussing aspects of forces resulting from wind stresses and structural stiffness, Fleming (18) suggested the solution was quite indeterminate and indicated that observations of actual building performance were required. But he observed further: "owners of tall buildings especially those very tall, are not friendly to observations for vibrations being taken. In most cases they refuse permission".

After some ten years of study into wind effects on buildings, Subcommittee No. 31 of the ASCE Committee on Steel of the Structural



Documents

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Division published its report (19). With respect to wind loadings it was recommended that a uniformly distributed load of 20 psf for the first 300 ft. above ground and increased by 2.5 psf for each additional 100 ft. in height should be used. For buildings normally considered airtight an internal pressure or suction of 4.5 psf was assumed acting normal to the walls and roof. In proportioning the wind bracing of tall buildings it was considered not necessary to divide the wind into pressure and suction effects. The subcommittee recognized that although walls and partitions may serve to lessen vibration and deflection, they should be ignored in strength calculations. For buildings having a high ratio of height to width provision should be made in the structural frame for the entire recommended wind load. Further, they recommended that more study and experimentation was necessary to determine a desired degree of comfort to the occupants insofar as building vibrations were concerned and until more understanding is available it was recommended that the maximum computed deflection of a building at the top where the structural frame was assumed to carry the entire lateral force should not exceed 0.002 of the height. It was also stated that for very high buildings, "this value represents not only a satisfactory occupancy basis but about the upper practicable limit of attainable stiffness without large additions to the girder sizes for wind effect only". Higher theoretical frame deflections could be tolerable as it was noted that some buildings of 20 to 25 stories in height had deflection indexes upward of 0.004 or 0.005 and performed satisfactorily, but it was cautioned that relaxation of the recommendation should come only as a result of additional experience not only from a human comfort point of view but also to guard against injury to external walls and internal partitions.

A task force of ASCE presented a comprehensive report (20) on the nature of wind and distribution of extreme winds in the USA. Equivalent static forces taking into account the shape orientation of objects were presented and extended to unsteady wind forces.

Specifically the committee recommended that the basic wind velocity be defined as the fastest-mile-of-wind reduced to a height of 30 ft. as recommended by Thom (21) for various geographical areas of the United States. Two basic winds were recommended: (i) a statistically derived probable period of recurrence of 50 yr. for permanent structures, and, (ii) a probable period of recurrence of 100 yr. for extremely wind sensitive buildings and those having a high degree of hazard of life and property. Recommendations to recognize the "unsteadiness" of wind forces and aspects of gust winds, vortex forces, drag and lift were also presented.

Additional information has been subsequently published by Thom (22) presenting basic wind velocities for various locations in the United States for recurrence periods of 2, 10 and 25 yrs.

The last decade has been most productive of activity in the study of dynamic wind effects and dynamic structural response (23,24, 25,26,27,28,29,30). Davenport (27) succinctly summarized the state of the art regarding wind loadings and wind effects on structures.

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CURRENT DEFLECTION AND SERVICEABILITY REQUIREMENTS

Current structural design practice to prevent undesirable effects of vibrations both for structural members and structures ranges from satisfying a simple span/deflection limitation or deflection index, to a full dynamic analysis.

Members--Serviceability requirements currently in use consist of either: (i) checking a computed deflection corresponding to a specified loading and restraint condition for a structural member having specific section properties against an allowable deflection, or, (ii) checking a members' span/depth ratio against a maximum allowable span/depth ratio. Satisfactory performance in terms of meeting the above types of limitation implies that no serious problem should arise with structural considerations such as plaster cracking, sagging floors, lack of fit of doors, etc., or with vibrations of floor systems resulting in annoyance from china rattling, etc., (31,32).

A multiplicity of Building Codes exist today and each contains its particular member serviceability requirements. Although it might be desirable to include in this report a survey listing all such requirements this has not been done because in many cases these serviceability requirements are very similar, and also, they are based on the recommendations of particular Standards centering around specific materials, as for example, The American Institute of Steel Construction, The American Concrete Institute, and The American Institute of Timber Construction. A very complete survey of code member serviceability requirements is available in a report dealing with structural deflections (33) for a large number of American building codes and including similar information for some European codes. Because the rate of introduction of structural serviceability requirements has been greatest in the area of reinforced concrete, for comparison purposes the discussion which follows dealing with concrete includes also the most recent requirements of the Comité Européen du Béton of 1970 (34).

For steel members the serviceability limitation given by the American Institute of Steel Construction (35) for deflection resulting from live loads to span, δ/L , is $1/360$ if the member supports a plastered ceiling; in general, "Beams and girders supporting floors and roofs shall be proportioned with due regard to the deflection produced by design loads". There is a further limitation specifying that the depth/span of a beam is to be less than or equal to the allowable bending stress, F_b divided by 800 in floors and $F_b/1000$ for roof purlins if they are not flat.

Serviceability requirements for structural members made of reinforced concrete are given by the American Concrete Institute (36). Two serviceability requirements are available for flexural members, (i) minimum thickness in terms of span length, and (ii) maximum allowable computed deflection in terms of span length. In the former the minimum thickness to span is specified for various end support conditions and, modifications are also presented to account for variations in concrete and the yield strength of the reinforcement. Table 2 presents the serviceability requirements of the 1971 ACI Building Code (36).



For reinforced concrete elements in service load ranges not only is it necessary when calculating deflection to account for a two material member which is partially cracked and has a complex situation with respect to area moment of inertia, but also for the time dependent deformations which occur as a result of sustained service loads. The guidance given by ACI regarding such calculations are summarized in Table 3.

The recommendations of the Comité Européen du Béton (34) regarding serviceability relative to deflections consist essentially of span/depth limitations such as: 10 for cantilever slabs or beams, 12 for rectangular or T beams, 25 for slabs, ribbed floors or hollow floors. Whenever span/depth limitations are less than cited above guidance is given, in terms of modulus of elasticity and moment of inertia, for the calculation of deflection. No guidance regarding limiting values for deflection to span is given. Recommendations are given for the beneficial effects of compression reinforcement and varying humidity effects in the prediction of deflections under prolonged loadings.

For structural members made of timber similar values for span/depth limitations as a measure of serviceability to those cited above are used. The American Institute of Timber Construction (37) recommends for floor beams that the span/depth serviceability limitation be the smaller of that corresponding to $1/360$ for live load and $1/240$ for dead plus live load under ordinary usage. For roof beams AITC has several recommendations and in all cases live load effects alone or total load effects are examined and the more severe limitation governs. Roofs supporting plastered ceilings have a δ/L limitation of $1/360$ and $1/240$ for live load only, and, total load, resp., and with no plastered ceiling the corresponding limits are $1/240$ and $1/180$.

With the recent development of light weight steel joist, concrete or gypsum plank type floor systems, vibrations in such systems have gained in importance. No specific requirements except those cited above exist for such floor systems however information and guidance is available for designers. The criteria shown in Table 4 (31) "control vibration effects due to foot impact by controlling the stiffness of the floor and hence its amplitude of vibration". It is also indicated that resonance problems resulting from humans will not occur if the natural frequency of the floor systems is greater than 5 cps.

An extensive study of such floor systems has been made by Lenzen and summarized in a state of art report (27). He noted no problems for spans less than 20 ft. For larger spans range of vibration varied from 2 to 15 cps with amplitudes of from 0.008 to 0.02 in. due to human "drop of heel" loading. Damping of the floor systems was observed to range from 3 to 5% but improved to well above 6% with inclusion of partitioning and mechanical services. In this range of vibration the human responds mainly to acceleration. If sufficient damping is present to limit vibrations to within 5 cps Lenzen observed "the human usually will not respond to the vibration", otherwise the response will be like for steady-state vibrations. To minimize vibration effects Lenzen recommended that "all beams and girders should be sized as though they were simple beams" since the dynamic stiffness of a continuous beam is the same as that of a simple beam. Addition-

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ally, artificial damping or reduced amplitude through the use of a thicker concrete slab will alleviate vibrations in floor systems.

Serviceability requirements for non structural elements relate primarily to the functioning and visual appearance of such elements. Exterior curtain walls must be stiff enough to resist excessive vibrations from wind gusts, and not deflect perceptibly when leaned against. It is recommended (25) that the deflection of curtain walls made of metal mullions sills and panels be limited to clear span/250.

With respect to glass, absolute serviceability requirements such as clearances of the order of 1/4 in. to provide for thermal expansion and contraction must be provided. Additionally glass must resist wind forces, and guidance to the design of glass panels, consisting of relationships between area and wind pressure for various thicknesses and adjusted to account for short and long dimensions and edge support conditions, is presented in References 25 and 38.

Structures--Current serviceability requirements for structures relate not only to physical building damage but also to annoyance and discomfort of occupants. Damage in buildings includes cracking of partitions, failure of cladding and glass, misalignment of elevator shafts, dynamic and static buckling, fatigue and possibly plastic collapse (23). Discomfort of occupants includes a variety of factors ranging from simple perception and annoyance with the vibration to motion sickness with headaches and dizziness, and extending to nausea, according to Reed, Hansen and Vanmarcke (27).

In spite of the complexity of this problem, the current serviceability requirements regarding lateral displacements, where available, involve simply the use of a limiting deflection index as a measure of serviceability. It should be indicated that lateral displacements of structures due to wind have two components: (i) a steady state, or nearly so, static deflection and, (ii) an oscillatory motion with respect to the steady static deflection. This factor is significant in recognizing the limitation of use of a simple maximum deflection index as a criteria for satisfactory structure performance.

In a state of art report, (27), Johnson stated "there is no established "drift" criteria for the design of buildings". He cited the general requirement of the 1970 Uniform Building Code that "Lateral deflection or drift of a story relative to its adjacent stories shall be considered in accordance with accepted engineering practice". He also indicated that the Los Angeles Building Code limits allowable drift under wind to 0.0025 of the building height.

Because of the paucity of information on drift limitations the information presented by Eligator and Nassetta (38) and summarized in Table 5 can be considered as typical of current engineering practice. In deflection calculations they indicated that stiffening effects of floors, and, interior and exterior walls are highly indeterminate and are neglected. Variations in the deflection index given above recognize the stiffness contributed by masonry exterior and interior walls and the lack of contribution to overall stiffness by metal and glass curtain walls. In their report (32) ACI Committee 435 presented recommendations for serviceability with respect to lateral building movement. They suggested the maximum deflection of a building

due to a five minute sustained wind load be limited to 0.002 of the building height, or, an absolute value of 0.15 in. offset per story height. The recommendations of the National Building Code of Canada (31) included a lateral deflection limitation of 1/250 to 1/1000 of the building height to prevent cracking of masonry and interior finishes.

In a summary report Le Messurier (27) indicates in recent years with the tightening of lateral deflection/height to 1/600 that "this was a settled issue". In his experience this criteria was satisfactory in buildings extending to 600 ft. in height. He next draws attention to the unsatisfactory performance of a new conventional building in 1970 in which the drift limit was 1/700, and finally concluded that "the ultimate issue is human comfort"; acceleration and damping were the critical parameters.

Throughout the last half century an effort has been made to determine parameters which would relate deflection and vibration of members and structures to human response. Spurr in 1930 (15) suggested that a parameter important in relating to human response was acceleration. He also felt that amplitude and frequency were significant and suggested "a factor of disturbance" as the product of amplitude and frequency. Another early study was that of Reiher and Meister (39) cited in (40). This study involved human response when subjected to steady state vibrations which were atypical to transient building vibrations. Figure 1 (40) shows the results of Reiher and Meister tests for steady state vertical vibration of a person standing. Goldman and von Gierke (41) also studied steady state vibration effects on humans; their results are shown in Fig. 2.

Chang (42) suggested that degrees of comfort of occupants can be defined in terms of the buildings' acceleration as follows: under 0.005 g - motion not perceptible, 0.005 to 0.015 g - threshold of perceptibility, 0.015 to 0.05 g - annoying, 0.05 to 0.15 g - very annoying, and over 0.15 g unbearable. Chang (43) later presented a plot of comfort limits as functions of amplitude and period shown as Fig. 3, and applied his criteria to assess the comfort response of four prominent tall buildings concluding that specific response is "in line with the experiences of the buildings occupants".

Davenport (23) introduced a statistical approach for design against wind effects. He summarized the deflection history of a structure in terms of a deflection occurrence curve which is shown as Fig. 4. Attention is drawn to the effects of building damping, and the envelope of maximum tolerable accelerations which are a function of annual occurrences of building deflection. Davenport (24) extended his statistical approach to include criteria for satisfactory performance and recurrence intervals in years for various factors affecting serviceability, as shown in Table 6. Figure 5 presents Davenports' envelope of design deflection limitations for tall structures as a function of the number of occurrences of these deflections per annum.

Chen and Robertson (44) investigated to establish perception thresholds of horizontal motion in frequency ranges applicable to tall buildings. Analysis and interpretation of the data was on a statistical basis and their findings were summarized on a plot shown as Fig. 6. They indicated such information "would be useful in evaluating a proposed building design with respect to comfort and the occupants".

Reed, Hansen and Vanmarcke (27) presented results of a

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study on human response to wind induced motions in tall buildings. They detailed classes of factors influencing human response as environmental motion cues, external manifestations and internal factors of response. Regarding environmental motion it is generally accepted that "a building is not supposed to move" and hence important factors in this class include the feeling of building movement, noise associated with differential movements of building components, fixture, and furnishing movements, and, perceived motion relative to the outside particularly under torsional movement and fluctuating tilt. External manifestations and internal factors discussed relate to human senses involved in perception of motion and the psychological and physiological effects resulting from these motions. The authors noted the need for testing for motion response in the "atmosphere" of a tall building since "the tall building is an individual entity producing its own environmental and psychological factors". Based on response of occupants in tall buildings subjected to perceptible motions the authors concluded "that people are sensitive to motion in tall buildings and do not consider perceptible motion acceptable unless it is a sufficiently "rare" occurrence".

In extending their previous work Reed, Hansen and Vanmarcke (45) presented the results of their investigation of human response to wind induced motions in tall buildings and a design criteria from an owners perspective, in the form of a plot shown as Fig. 7.

DISCUSSION

A study of historical developments in the building process and deflection requirements related to vibrations shows a parallel exists between these topics. As the building process changed and improved so too did the requirements for satisfactory structural performance. The start of the use of a deflection requirement as given by a limiting span to member depth ratio, reflected unsatisfactory performance both from a visual and vibratory point of view. Continued experience and introduction of new materials and building processes during the nineteenth century resulted in some adjustments to this deflection serviceability requirement. This type of requirement has persisted and is still in use currently. For a material like concrete only recently, since the use of the ultimate strength design procedure, has a deflection requirement been introduced. Generally, the phenomenon of excessive deflections is a fairly recent one, having its beginnings in the late 1950's, and reflects the trend towards higher strength materials, less stiff materials, shallower members, improved design procedures recognizing interaction between components, and more lightly clad buildings having reduced damping.

With regard to serviceability limitations currently in use for controlling static structural member deflections it may be concluded that the form of limitation δ/L or its related counterpart, L/d , is satisfactory and convenient from a design point of view. Such limitations have a rationale and are not merely historical accidents that work. Limiting δ/L to a constant in effect limits the maximum strain in bending that can occur in the extreme fiber regardless of the material used. It should be noted that whereas the service load working stress varies significantly among the various materials the

range of corresponding strain is considerably narrower; in fact, the range is almost identical for steel and wood. For concrete, if time dependent deformations are included, the range of working strains would be only slightly greater than that for wood or steel. Because of this similarity of working levels of strain a single limitation, such as δ/L equal to $1/360$, has served for a long time as a reasonable measure of satisfactory performance with respect to static structural deflections. Similarly, the L/d requirement has a rationale which in its derivation is governed by a specific level of stress at service loads for a given structural member.

The advisability of extending the use of these static serviceability limitations to the control of dynamic structural member deflections is open to question. The only argument in its favor is recognition that historically it seems to work. There is considerable information available now that indicates parameters other than deflection are significant, particularly for human response to vibration. Acceleration, rate of change of acceleration and even velocity have been shown to be more important than displacement for human response considerations. In studies of floor systems the beneficial effects of damping have been indicated insofar as perception or acceptance of vibrations is concerned. More experimentation and study is necessary to clearly delineate the critical parameters and limitations for satisfactory performance.

The problems incurred with structural deflections of tall buildings are similar to those cited above for structural members. The considerations for serviceability with respect to deflections are intimately connected with wind loading requirements. Over the last half century a deflection index limitation has existed as a measure of satisfactory performance. In a tall building such a limitation may provide against structural or non structural damage however the amount of actual displacement to cause damage may be far in excess of that which can be tolerated by human occupants in terms of building accelerations or rates of change of acceleration. The use of lighter curtain wall materials and extensive use of glass together with lighter interior partitions all contribute towards a higher degree of concern for the satisfactory performance of a tall building. Clearly, severe limitations exist in the use of a deflection index as a measure of performance relative to vibrations. Recent studies by various investigators cited in the previous section suggest that acceleration is the important parameter with respect to human performance. Furthermore, the latest studies indicate that human acceptance of motion or its effects in a tall building varies depending on the severity and frequency of the storm wind loading.

SUMMARY

This report presents a review of historical developments and current deflection requirements as they relate to vibrations in structural members and buildings. To maintain a perspective on deflection limitation developments, a review of the building process itself is presented.

Two classes of deflection requirements are studied. In the case of members, transient vibrations resulting from human occupancy

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or movement of live loads and their effects are discussed. In the case of buildings, the vibrations of concern are those induced by wind.

Deflection limitations as a measure of serviceability for members and buildings are cited and discussed. Generally, for both types of limitations the applicability or short coming of each of these limitations is indicated. A most significant factor in extending a deflection limitation or index to account for all aspects of satisfactory serviceability is recognition, in the case of human comfort considerations, that acceleration is more significant than deflection.

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TABLE 1

Slab RequirementsEarly 1900's - Min. slab thickness6 in. or $L/32$ (floors) or $L/40$ (roofs)

$$t/L = 0.019\sqrt{w}$$

where w is total loading.1920 - 1940 - Min. slab thicknessfloor: $L/32 \geq 6$ in.roof: $L/40 \geq 5$ in.

$$(t-1)/L = 0.02\sqrt{w}$$

1941 - 1951 - Min. slab thickness $L/40$ with drop panels $L/36$ without drop panels1956 ACI Code - Min. slab thickness $L/40$ with drop panels ≥ 4 in. $L/36$ without drop panels ≥ 5 in.

$$t_1 = 0.028 L \left(1 - \frac{2c}{3L}\right) \sqrt{\frac{w'}{f'_c/2000}} + 1 \frac{1}{2} \quad \text{no drop panel}$$

and

$$t_2 = 0.024 L \left(1 - \frac{2c}{3L}\right) \sqrt{\frac{w'}{f'_c/2000}} + 1 \quad \text{with drop panel}$$

1971 ACI Code (Ref. 36) - Min. slab thickness

For slabs without beams or drop panels - 5 in.

For slabs without beams, but with drop panels - 4 in.

For slabs having beams on all four edges with a value of

 α_m at least equal to 2.0 - 3 1/2 in.

$$h = \frac{\frac{1}{n} (800 + 0.005f_y)}{36,000 + 5000\beta [\alpha_m - 0.5 (1 - \beta_s) (1 + \frac{1}{\beta})]}$$

but not less than

$$h = \frac{\frac{1}{n} (800 + 0.005f_y)}{36,000 + 5000\beta (1 + \beta_s)}$$

The thickness need not be more than

$$h = \frac{\frac{1}{n} (800 + 0.005f_y)}{36,000}$$

TABLE 2

Serviceability Requirements ACI Code (Ref. 36)

Minimum Thickness of Beams or One-Way Slabs
Unless Deflections are Computed

Member	Minimum thickness, h			
	Simply supported	One end continuous	Both ends continuous	Cantilever
Solid one-way slabs	1/20	1/24	1/28	1/10
Beams or ribbed one-way slabs	1/16	1/18.5	1/21	1/8

Maximum Allowable Computed Deflections

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to non-structural elements likely to be damaged by large deflections	Immediate deflection due to the live load, L	1/180
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections		1/360
Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections	That part of the total deflection which occurs after attachment of the nonstructural elements, the sum of the long-time deflection due to all sustained loads and the immediate deflection due to any additional live load	1/480
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		1/240

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TABLE 3

Deflection Computations ACI Code (Ref. 36)

Immediate Deflection

Where deflections are to be computed, those which occur immediately on application of load are computed by the usual methods using:

Modulus of Elasticity, $E = W^{1.5} 33 f'_c$

where W is the weight of concrete.

$$I_e = \left(\frac{M_{cr}}{M_a} \right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a} \right)^3 \right] I_{cr}$$

where

$$M_{cr} = \frac{f_r I}{y_t}; \quad f_r = 7.5 \sqrt{f'_c}$$

Long-time Deflection

Unless values are obtained by a more comprehensive analysis, the additional long-time deflection shall be obtained by multiplying the immediate deflection caused by the sustained load considered by the factor

$$[2 - 1.2 (A'_s/A_s)] \geq 0.6$$



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TABLE 4

Summary of Maximum Deflection/Span Ratios in Standards

	Timber	Reinforced Concrete	Structural Steel	Structural Aluminum
Roof or floor members supporting plastered ceiling, partitions, etc.	1/360 ⁽¹⁾	1/360 ⁽²⁾	1/360	1/360
Floor members (not supporting plastered ceiling, partitions, etc.)	1/180 ⁽¹⁾	1/360 ⁽²⁾	1/320 ⁽⁴⁾	1/200
Roof members (not supporting plastered ceiling, etc.)	1/180 ⁽¹⁾	1/180 ⁽³⁾	1/180 ⁽⁵⁾ or 1/240 ⁽⁵⁾	1/180
Wall members	1/180 ⁽¹⁾	---	---	1/180

Notes:

- Deflection under live load only unless otherwise noted.
- (1) Deflection under dead plus live load. Modulus for calculation based on short term test but there is a warning clause on creep deflection.
 - (2) Immediate live load deflection plus creep deflection due to dead load plus permanent live load.
 - (3) Immediate live load deflection.
 - (4) There is a warning clause on vibrations.
 - (5) 1/180 applies to sheet metal or elastic membrane roof cover and 1/240 to asphaltic built-up roofs. There is a warning clause on ponding.

from Ref. 3

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TABLE 5

Deflection-index Guide

Type of Building	Type of construction		Exposure	Code wind requirements	Deflection index
	Walls	Floors			
Office	Curtain wall	Steel, protective ceiling	Min.	Moderate	0.0025
Hotel	Masonry	Concrete	Max.	Extreme	0.0025
Office	Masonry	Steel, concrete encased	Avg.	Moderate	0.003
Residence	Masonry	Concrete	Min.	Extreme	0.0025
Office	Curtain wall	Steel, sprayed F.P.	Max.	Minimum	0.0015

from Ref. 38

TABLE 6

Typical Criteria For Design of Tall Buildings Against Wind

Unserviceability Symptom	Acceptance Criteria	Recurrence Interval: Years
1) Collapse	Safety factor = 1.1	500
2) Cracking of masonry and finishes	Maximum def'n. $< \frac{1}{250} \rightarrow \frac{1}{1000}$ of height	30
3) Windows and cladding: (a) perceptible deflections; (b) breakage	(a) dependent on size of light, colour and type of glass (b) < 1 breakage per building	(a) 5 (b) 10
4) Fatigue	Cumulative damage $< 100\%$	500
5) Comfort of occupants occupants	Maximum acc'n. $< .5 \rightarrow 1.5\%$ g	10

from Ref. 24

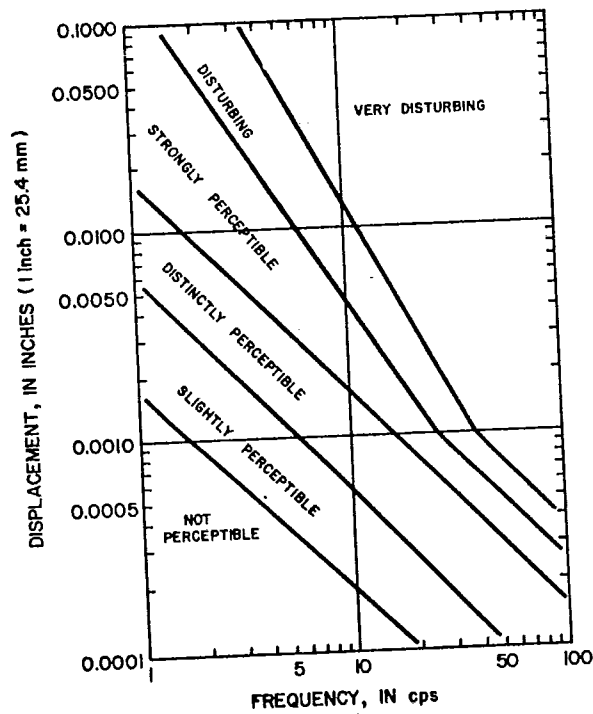


Fig. 1--Human response to steady-state vertical vibration from Reference 40

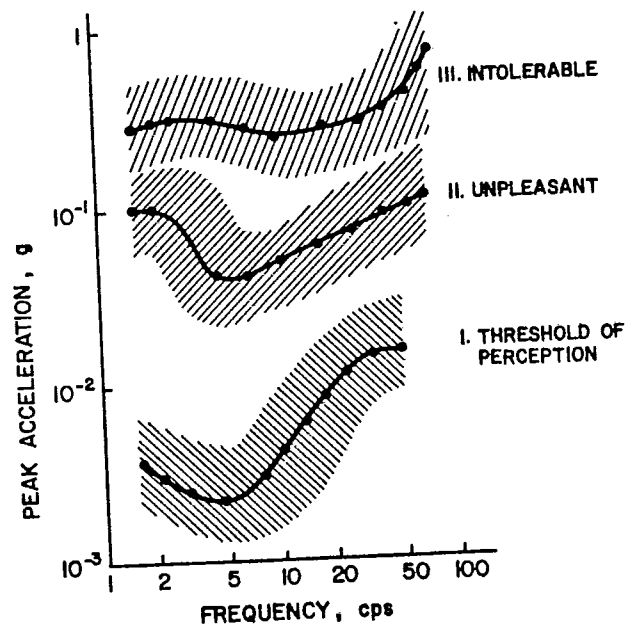


Fig. 2--Peak accelerations at various frequencies from Reference 41

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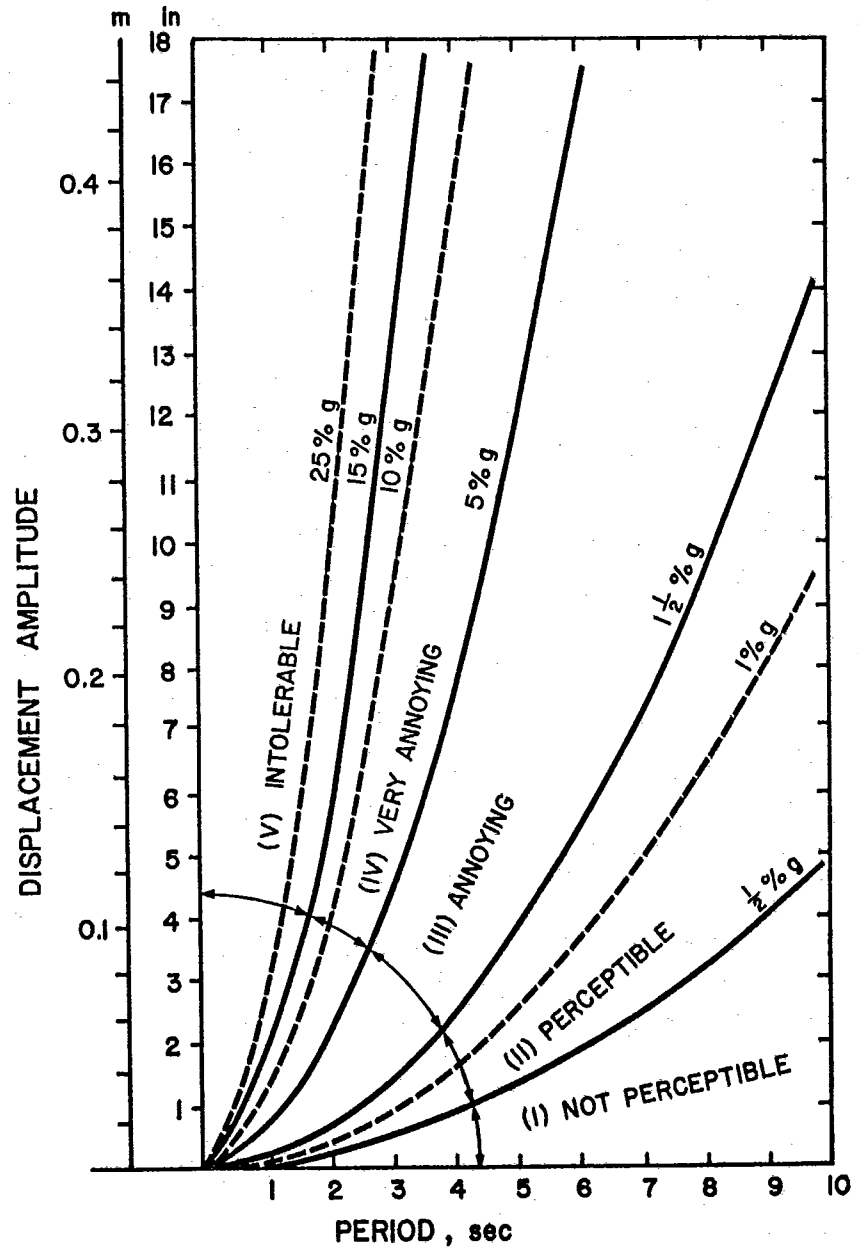


Fig. 3--Recommended human comfort curves from Reference 43

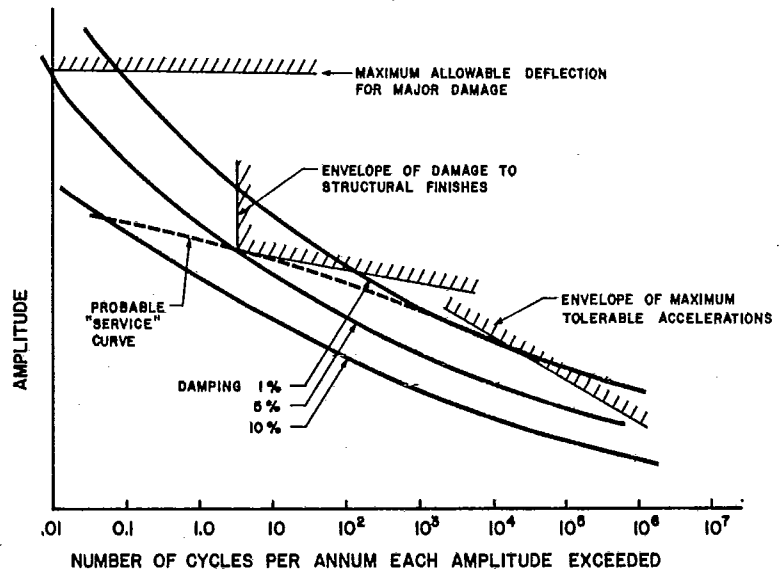


Fig. 4--Amplitude--occurrence curve from Reference 23

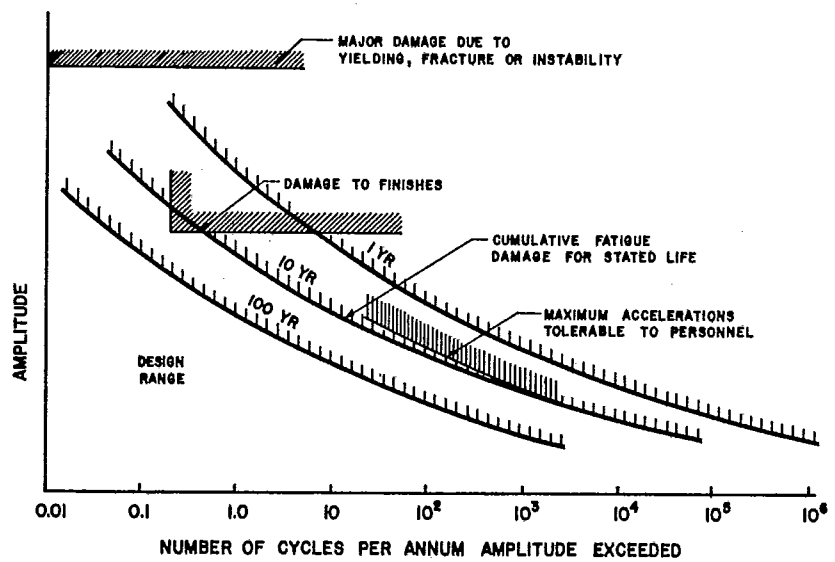


Fig. 5--Envelope of design limitations from Reference 24

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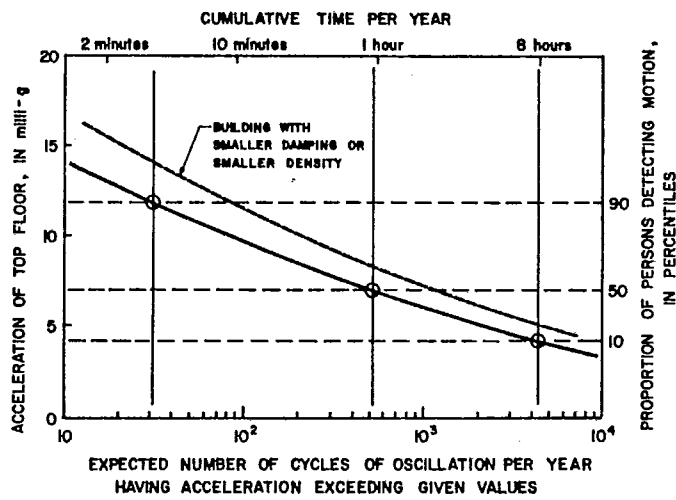


Fig. 6--Dynamic response recurrence diagram of building from Reference 44

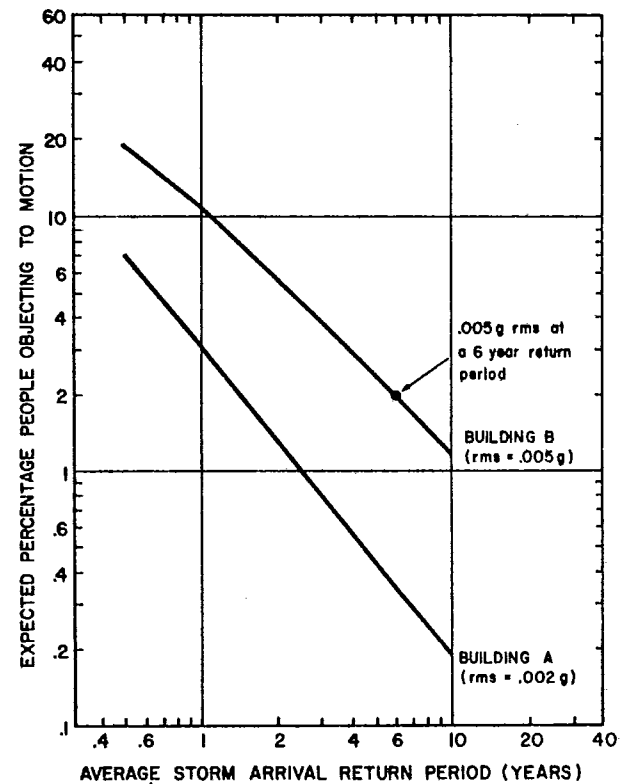


Fig. 7--Criteria selection from Reference 45

SP 60-3

Development of Model Performance Criteria for Deflection of Concrete Floors

By Robert A. Crist and James A. Shaver

Synopsis: Serviceability performance criteria for floor systems are discussed in terms of their static and dynamic components. Traditional static stiffness criteria are discussed along with a review of their strengths and weaknesses. Model performance criteria for serviceable floors are developed from a vibration viewpoint. A new approach for future floor vibration criteria is described.

Keywords: concrete slabs; deflection; dynamic loads; floors; human factors engineering; mathematical models; performance; serviceability; static loads; vibrations.

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Development of Model Performance Criteria for Deflection of Concrete Floors

INTRODUCTION

This paper covers the development of performance criteria for structural floor systems with the focus being on that part of the criteria which is related to serviceability. The paper is based on a portion of the research done in a broader study* (Structural Deflections [1])¹ at the National Bureau of Standards.

The design of floors for serviceability has been in existence for many years. A serviceable floor is one that meets the needs for which it is intended for everyday use. Seldom, if at all, does a serviceability design requirement equal a stress design requirement, i.e., a life safety requirement. Floors that are safe are not necessarily serviceable; however, floors that are serviceable generally are safe. Safety here refers to collapse and endangering human life.

Deflection criteria for the design of floors can appear in a hierarchy (figure 1) beginning with a performance statement. The examples in figure 1 converge from the generality of a performance statement to specific technical engineering publications, research publications, and engineering texts and are the types of statements applicable to occupant comfort of floors. The differentiation made in the

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¹ Numbers in brackets refer to literature references listed in Section 7 of this report.