Theme 5: Response to service actions

THEME 5

RESPONSE TO SERVICE ACTIONS

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Performance Requirements for Building Structures in Post Terminals

Conditions de fonctionnement des structures de centres de tri postaux

Betriebsbedingungen für Gebäudekonstruktionen von Post-Teminals

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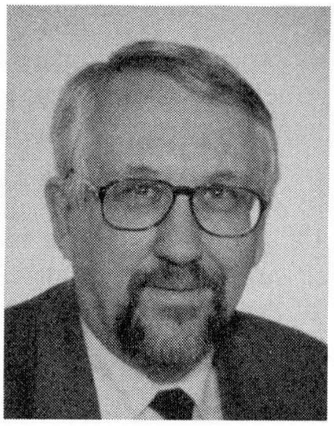

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SUMMARY

This article gives a general description of the requirements for building structures in post terminals, considered both from current user needs and also the longer perspective of real estate development. The philosophy is that post terminals should be considered as designed for light industry yet be able to facilitate a certain amount of change during their life span. The examples have been selected from existing Sweden post terminals.

RESUME

Cet article présente une description des exigences techniques pour la construction des structures de centres de tri postaux. Ont été pris en considération le point de vue du client usager et celui des proprétaires afin d'optimiser l'investissement. Ces centres de tri doivent être conçus comme des bâtiments d'industrie légère et pouvoir être adaptés à de nouveaux besoins. Les exemples cités ont été pris parmi les centres de tri existants de la Poste Suédoise.

ZUSAMMENFASSUNG

Dieser Artikel vermittelt eine allgemeine Beschreibung der Bedingungen für Gebäudekonstruktionen von Post-Terminals im Hinblick auf die Erfordernisse für die derzeitige Nutzung sowie aus der Sicht einer längerfristigen Nutzung. Grundgedanke hierbei ist die Gleichsetzung der Post-Terminals mit Gewerbegebäuden für Kleinindustrien, die während ihrer Lebensdauer einer Reihe von Umbauten und Änderungen ausgesetzt werden können. Die Beispiele stammen von Terminals der Schwedischen Post.

1. THE FUNCTION OF POST-TERMINALS

This paper deals mostly with the demands on building structures when handling mail in post-terminals. Many of the demands are similar to those placed by light industry with assembly lines. This is because of the frequent use of mechanisation in the mail-sorting process.

The sorting of letters in large terminals is a process involving automated sorting machines, manual sorting, various forms of conveyors and even traffic with fork-lift trucks. This constitutes a combination putting specific demands on the properties of the load-bearing structure.

The sorting of parcels is a heavier process and demands larger installations able to accommodate larger parcel-sorting equipment, often 200 metres long, combined with the use of fork-lifts and other equipment for the transportation of parcels to and from the loading-bays for road and ramps for rail transport.

As with any other industrial activity, mail-sorting is often subject to changes in methods, and equipment used. The flow of mail varies in itself, changes regarding types of product and volume of letters or parcels. The layout of the machinery and the manual work-stations is not permanent and regular redeployment occurs.

Postal operations are very much time-restricted. The mail must be processed within a given period to meet the time schedules of road, rail and air transport. The sorting terminals must not, in any way, impede the flow but rather enhance it by virtue of rational planning and effective structuring.

It is preferable not having to transport mail to different levels within the building for processing. A single-storey construction is ideal when considering internal logistics. However the constaints on the deployment of sorting-centers within the infrastructure of cities prevents such optimal solutions.Available sites are too small when the need for good connections with rail junctions, major highways and yet close proximity to city centres are necessary parameters. Therefore, multi-storey constructions for postal terminals are common. The dimensions of main sorting-terminals in Sweden vary between 20,000 sq.m. - 100,000 sq. m.

Parcel-sorting operations are usually located on the ground floor while letter-sorting can take place on first or second floors. On floors above the "production" levels, are the office areas and convenience facilities - locker-rooms, canteens and rooms for physical fitness etc. The terminals are equipped with loading-bays for road and ramps for rail transport, both of which may be open or covered areas.

The existing and future sorting equipment, conveyors and overhead-rail conveyors have determined room dimensions and ceiling heights. Between storeys in large terminals, the floor to floor measurement is 4.5 - 7 metres. Halls with high ceilings are needed for parcel-sorting.

Therefore one talks about "hall solutions" of the sorting areas. Sorting activities go on almost continuously round the clock, with peaks during the evening, night-time and early morning. Very few moments are free from postal-processing, which means that very little time is available in which to carry out repairs that disturb the sorting operations.

2. LOAD-BEARING CAPACITY

Demanded load-bearing capacity varies according to the type of postal activities. In Sweden Post we deal with the following range of load characteristics which can be a combination:

A. Pallets stacked in 3 layers, paths for fork-lifts, load from installed machinery.
B. Post-containers handled with fork-lift, load from installed machinery.
C. Point loads from equipment installed in a 3m. x 3m. pattern, load from installed machinery.

D. Point loads from machine platforms, load from post-containers and other installed machinery.
E. Pallets stacked in 2 layers with paths for fork-lift, load from installed machinery.
F. Pallets in single layer with paths for fork-lift, load from installed machinery.
G. Manual handling of post-containers, load from installed machinery.

3 .EVENNESS OF FLOORS

Level floors in postal terminals is a must as the mail-containers must stand still and not roll away. The maximum permissible slope, at any point, is 1/200, otherwise there is a risk of runaway containers. This can be especially dangerous on loading-docks where uncontrolled containers could roll over bay limits or otherwise constitute a hazard.

Evenness is also a prerequisite for the successful installation of post-handling equipment such as automatic letter-sorting machines.

4. EXPANSION JOINTS

Expansion joints are usually a problem due to the difficulty of making perfectly level joints in floor surfaces. Uneven joints cause vibration with fork-lift equipment.

5. CRACKS IN CONCRETE FLOORS

Wear and tear on floors during post-handling is considerable, especially from fork-lifts and mail containers. Floors should be made as durable as possible during construction as it is practically impossible to make repairs during round-the-clock postal activities.

The floor surface should be hard in itself or treated in such a way that it does not emit dust which can disturb the function of finely-adjusted sorting equipment. The surface should consist of hard concrete, preferably vacuum-concrete and can be given an epoxi coating. There must be no filling in the concrete, as filling materials would not stand up to the pressure and cracks will appear. Single-course floors with vacuum treatment have proven to be the best for this type of activities. In zones where manual sorting is predominant, it is necessary to have a softer material on top of the concrete, but the construction of the floor slab should be the same as in zones with harder wear as one never knows how the layout will be in the future.

6. DEFORMATIONS

Floor surfaces in halls designed for postal production should always conform to the following specifications:
The limit of decline is fixed as the floor-slope should, at no point over the entire length of the floor, exceed1/200. The incline tends to increase at the end of the supporting beams, especially when slabs of pre-stressed concrete are used.
Therefore the total deformation is limited to L/600.

7. SPAN

The span of the bays must be determined on the basis of layouts. Columns should interfere neither with the machinery nor with the work-positions with manual handling. Certain layouts are often subject to revision, especially with letter-sorting, this due to new techniques and the varying types of post being handled.

To facilitate good layouts, not impeded by columns and to ensure good flexibility for future layouts, the spans should be large.

The minimum span-width is 9 metres. For letter-sorting, 12 - 15 metres are often required. Even larger spans are necessary with parcel-sorting, between 20 - 25 metres. Sorting-halls of this type are ideally single-storey constructions, which can be combined with multi-storey building containing personnel facilities and office areas. Solutions with extremely large spans for parcel-sorting halls do exist, e.g. a single span of 30 - 40 metres instead of two or three bays as in other cases.

8. FLEXIBILITY

The larger spans meet the need for alternative use of the premises. The real estate agency of Sweden Post must have as a principle, that all post-sorting spaces should be able to be used in an alternative way, preferably for light industry or distribution purposes. This is due to the necessity of retaining and even increasing the market value of the property.

The structures should normally be built in such a way that horizontal and even vertical extensions are possible. This is to cater for a demand for increased space for postal activities on the one hand, and to maximise the use of development rights on the other.

On prime sites, where development rights are usually a restriction regarding height, it is, for economical reasons, necessary to reduce the height of the floor structures. This in combination with larger spans tends to result in weak floor constructions that might easily vibrate when using fork-lifts, machinery etc.

Extremely restricted sites in city centres can necessitate the use of steel columns instead of concrete, in order to reduce the area of the columns and utilize existing space effectively.

9. STIFFNESS OF FLOOR STRUCTURES

In some "first generation" post terminals, e.g. in the 100,000sq.m. Tomteboda Terminal in the Stockholm area, the structure of the upper floors are relatively weak. In this case this is due to the following reasons: long spans of 12 x 16.8 metres, and the combined steel and filigree-concrete construction of the floors. The weakness has the effect that fork-lift traffic causes vibrations in the floor that spread through the floors to working positions with manual post sorting, work-stations with automatic sorting machines, video-coding desks and to traditional office areas. In some areas the vibrations felt by personnel are certainly above an acceptable level, especially in areas where fork-lifts drive over uneven expansion joints. This feeling of insecurity experienced by personnel due to the vibrations transmitted via the floor structure is to be avoided. Therefore efforts must be made to erect structures more rigidly, and do not vibrate yet still meet the requirements regarding span and other desired properties, such as height of floor structure, decline etc.In this field there is a demand for still more research in order to attain criteria regarding vibration and its effects on the well-being of those employed in industrial environments.

10. HOLES IN FLOORS

In earlier projects for post terminals, one assumed the future would call for holes in the floors for the installation of various equipment etc. This has not really been the case, at least not to the extent anticipated. Still, it does occur that holes of considerable dimensions have to be made. This should be possible to accomplish without damaging the surrounding structure and can in fact be carried out successfully with the aid of e.g. lintels and need not be accommodated in any way causing increased investment costs in the eary stages.

11. ECONOMIC LIFE-SPAN OF STRUCTURES

In modern buildings, the cost of load-bearing structures does not represent a major investment. Still it is the properties of the structure that constitutes the current value of the building and the future value of the property while even determining the possible future uses of the structure. The structure is also the most durable part and should be possible to use to the fullest extent, even following a major conversion of the building for some purpose other than the original.

12. QUALITY ASSURANCE

The properties of structures should be considered with great care and it is also necessary to verify if any of these properties are lost during the building process. Therefore we work with quality control on these types of projects, during the whole process from programming to planning, design and the actual construction of the project. This has meant that a large number of mistakes and even some serious faults in the structures have been detected and rectified.

The effort and money spent on quality-management has been worthwhile indeed . Especially worth mentioning is the effect quality-management has on the clarity in programming and planning, also the verification of the qualities of building materials, prior to their being installed - thus making them impossible to inspect -.This type of successive inspection and verification of quality is necessary to ensure that a structure will meet the demands that may be placed on it at some time in the future.

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Preload Analysis of Reinforced Concrete Flexural Members

Influences des précharges sur les éléments en béton armé sollicités à la flexion

Vorbelastungseinflüsse auf biegebeanspruchte Stahlbetonbauteile

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SUMMARY

An analysis is made of the stress and strain in reinforced concrete flexural members at the preload stage under combined effects of temperature variations and shrinkage. Typical results of such analysis are presented in this article. A «prestressing analogy» is proposed and used by the author to provide a simplified concept of approach to the evaluations of the time dependent stresses in the members up to specified stages of durations of the external influence. The severe effects of temperature and shrinkage can also be opposed or «balanced» by imposing reverse prestressing effects determined from this analogy.

RESUME

L'article analyse les contraintes et les déformations induites dans les éléments porteurs en béton armé sollicités à la flexion, à l'état de mise en charge initiale par l'action combinée des variations de température et du retrait. L'auteur fournit quelques exemples typiques relatifs à cette analyse. Il propose une analogie avec la précontrainte et en déduit un concept simplifié, en vue de déterminer les contraintes en fonction du temps ainsi que les durées spécifiques des influences externes. A partir de l'analogie proposée, il est possible de compenser les effets considérables de la température et du retrait par une précontrainte agissant en sens contraire.

ZUSAMMENFASSUNG

Spannungen und Dehnungen in biegebeanspruchten Stahlbetontraggliedern werden im Stadium der Vorbelastung durch kombinierte Einwirkung von Temperaturschwankungen und Schwinden analysiert. Dafür werden einige typische Beispiele gegeben. Der Autor schlägt eine Analogie zur Vorspannung und ein vereinfachtes Konzept vor, um zeitabhängige Spannungen bis zu spezifizierten Einwirkungsdauern zu bestimmen. Insbesondere können aufgrund dieser Analogie erhebliche Temperatur- und Schwindeffekte durch entgegengesetzt wirkende Vorspannung ausgeglichen werden.

INTRODUCTION

Preload serviceability performance of reinforced concrete has seldom been considered in the design. The initial state of stresses caused by drying shrinkage and temperature variations have been frequently considered as secondary and hence negligible. In a large number of cases it is reasonable to ignore this initial state of stresses and performance since shrinkage may or may not act along with the effect of temperature variations. However in many not unusual cases, for example, in deep raft foundations or reinforced concrete walls under rigid end or side constraints, the preload effect of shrinkage and temperature can cause excessive cracking or warping. Hence affecting the serviceability performance of the structural members in the subsequent, loading, stage. The inadequacy of an analytical treatment on the time varying behaviour of reinforced concrete structure with particular consideration given to preload shrinkage and temperature effects has been mentioned in a few separate literatural surveys viz., by the ACI Committee 435(1) and by the unified Code Committee (2,3).

BASIC CONSIDERATIONS

The worst effect in preload analysis is to consider both shrinkage and the variation in temperature acting in unisón, i.e. causing longitudinal compressive strains in an unloaded reinforced concrete member. The longitudinal steel reinforcement provides some restraint to shortening so that internal stresses are induced in the concrete and steel. In a reinforced concrete flexural member which contains a preponderance of 'tensile' steel reinforcement near one face, the unsymmetric restraint provided by the longitudinal steel results in a non-uniform distribution of concrete stress and strain. The concrete tensile stresses vary from a maximum value at the face near the tensile steel to a minimum value at the opposite face, whereas concrete compressive strains vary from a minimum in the face near the tension steel to a maximum in the opposite face. The phenomena produce not only longitudinal shortening but also warping to the members. Such warping affects the long term performance of reinforced concrete flexural members particularly slender beams and slabs. Furthermore, when deformation is restrained, stresses developed and whenever concrete is stressed, creep occurs. Similarly, whenever stress is maintained over a constant deformation, it relaxes in the time course. All these bring about a continuous adjustment within such a member for self-equilibrium against the exterior influence.

PRELOAD ANALYSIS OF STRESSES AND STRAINS IN REINFORCED CONCRETE BEAMS

Consider a doubly reinforced concrete beam of rectangular section of width x depth = bxa and with compressive and tensile steel areas of As' and As which are located respectively at d' and d from the topmost fibre. Assuming that at this stage there is no external load acting on the member except that the beam is being subjected to the combined effects of temperature and shrinkage acting in unity (i.e. causing contracting to the member dimensions). At this stage it is further assumed that due to such combined action, the section remains uncracked and that the total strains and stresses are distributed linearly across. Such beam and section together with the strain and stress distributions and the resultants of forces are shown in Figure 1. The equilibrium of horizontal forces and bending moments (taken w.r.t fibre 1) in the section at time t leads to the formation of the following equations (with $\sigma 1$, $\sigma 4$, $\epsilon 1$, $\epsilon 2$, $\epsilon 3$, $\epsilon 4$ all time dependent qualities)

$$(\sigma_1+\sigma_4) + pE_s\,(2\beta\epsilon_2 + 2\beta r\epsilon_3) = 0 \qquad [1]$$

$$\sigma_1+2\sigma_4 + pE_s\,(6\beta^2\epsilon_2 + 6\beta\delta r\epsilon_3) = 0 \qquad [2]$$

where p = As/bd, r = As'/As, β = d/a and δ = d'/a

The linear distribution of strains allows $\epsilon 2$ and $\epsilon 1$ to be expressed in terms of $\epsilon 1$ and $\epsilon 4$ i.e.

$$\epsilon_2 = (1-\beta)\,\epsilon_1 + \beta\epsilon_4 \text{ and } \epsilon_3 = (1-\delta)\epsilon_1 + \delta\,\epsilon_4$$

By substituting the above expression into Eqs.[1] [2] $\sigma 1$ and $\sigma 4$ can be expressed as,

$$\sigma_1 = pE_s\,(Q_1\epsilon_1 + Q_2\,\epsilon_4) \qquad [3]$$

$$\sigma_4 = pE_s\,(Q_3\epsilon_1 + Q_4\,\epsilon_4) \qquad [4]$$

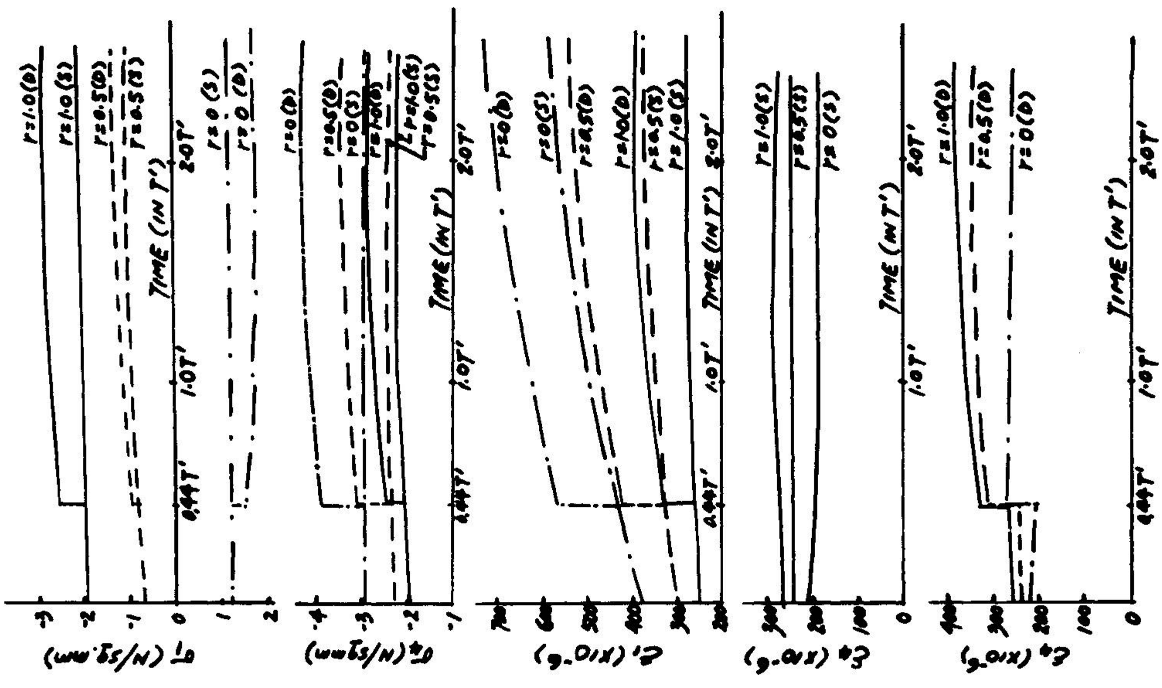

FIG.3: STRESSES AND STRAINS IN SINGLE(S) AND DOUBLE(D) STAGE PRELOAD ANALYSIS DUE TO COMBINED TEMPERATURE AND SHRINKAGE INFLUENCE

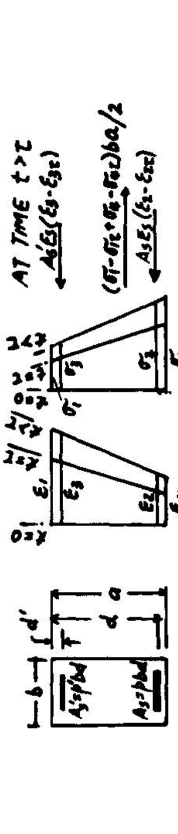

FIG.1: DEVELOPMENT OF TIME DEPENDENT STRAINS AND STRESSES DUE TO TEMPERATURE AND SHRINKAGE EFFECTS

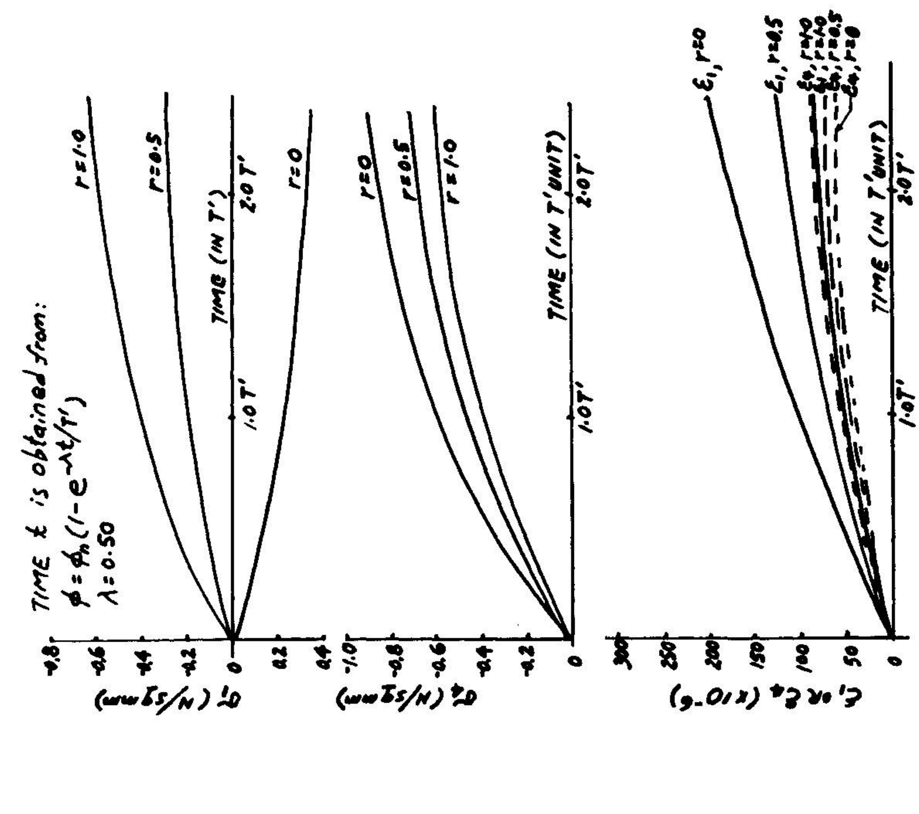

FIG.2: STRESSES AND STRAINS DUE TO SHRINKAGE ONLY

where $Q_1 = -2\beta[(1-\beta)(2-3\beta) + (1-\delta)r(2-3\delta)]$
$Q_2 = -2\beta[(2-3\beta)\beta + (2-3\delta)r\delta]$
$Q_3 = 2\beta[(1-\beta)(1-3\beta) + (1-\delta)(1-3\delta)r]$
$Q_4 = 2\beta[(1-3\beta)\beta + \delta)1-3\delta)r]$

The magnitudes of the time dependent strains $\epsilon 1$ and $\epsilon 4$ depends not only on the shortening caused by temperature and shrinkage but also on the sustained stressed developed at the same levels due to restraints provided by the reinforcing bars. The relationship between strain and stress in the concrete has been studied by Trost (3) and Bazant (4,5).

On basis of the principle of superposition in which an ageing coefficient has been incorporated for a range of situations involving stress relaxation histories the general form of this relation can be expressed as,

$$\epsilon = \sum_{i=1}^{n} \frac{\sigma_i}{E_i}[1+X_i\,\phi_i(t)] + X_i\,\phi_i(t)\frac{\epsilon_{sn}}{\phi_n} + \alpha\,\Delta T_i \qquad [5]$$

in which ϵ, σ, and X are all functions of age τ and time t. In the above expression X is the ageing coefficient which is a function of the relaxation of concrete and ϕ is the creep function. ϕ is related to ϕn which is the long term creep value.

In a one stage development the shrinkage and temperature variation are assumed commencing from one age and applying Eq.[5] for concrete strains at levels 1 and 4 in the section, the following equations are obtained,

$$\epsilon_1 = \delta_1\,[1+X\,\phi(t)\,]\,/\,E_o + X\,\phi\,(t)\,\epsilon_{sn}\,/\,\phi_n + \alpha\,\Delta T \qquad [6]$$
$$\epsilon_4 = \delta_4\,[1+X\,\phi(t)\,]\,/\,E_o + X\,\phi\,(t)\,\epsilon_{sn}\,/\,\phi_n + \alpha\,\Delta T \qquad [7]$$

By substituting the above equations into Eqs.[3],[4] and by rearrangement, a pair of simultaneous equations expressing $\sigma 1$ and $\sigma 4$ are obtained, i.e.

$$\sigma_1 = np[X\phi(t)E_o\epsilon_{sn}/\,\phi_n+E_o\alpha\Delta T][(Q_1+Q_2)\{1-npQ_4(1+X\phi(t))\}+(Q_3+Q_4)npQ_2(1+X\phi(t))]/R \qquad [8]$$
$$\sigma_4 = np[X\phi(t)E_o\epsilon_{sn}/\,\phi_n+E_o\alpha\Delta T][(Q_1+Q_2)npQ_3\,\{1+X\phi(t)\}+(Q_3+Q_4)\{1-npQ_1(1+X\phi(t))]/R \qquad [9]$$

in which $R = [1-npQ_1\{1+X\phi(t)\}][1-npQ_4\{1+X\phi(t)\}]-n^2p^2Q_2Q_3\{1+X\phi(t)\}^2$

In the above expressions the creep function $\phi(t)$ represents the time development of creep in concrete. $\phi(t)$ can thus be used to replace time in an one to one correspondence type of evaluations for the tensile stresses $\sigma 1$ and $\sigma 4$ in the concrete section. The signs for $\sigma 1$ and $\sigma 4$ are generally evaluated in negative values (representing tension). By substituting $\sigma 1$ and $\sigma 4$ values obtained from Eqs.8,9 into 6,7 the time dependent concrete strains $\epsilon 1$, $\epsilon 4$ caused by the effects of temperature and shrinkage can thus be obtained. The time varying curvature can thus be evaluated ie. $\rho = (\epsilon_1 - \epsilon_4)\,/\,a$

For a simply supported beam if the self weight is negligible and if the curvatures in all other sections are developed in identical manner, the warping deflection at mid-span at time t is thus $\Delta = \rho \ell^2/8 = (\epsilon_1-\epsilon_4)\,\ell^2/(8a)$

If further stages of changes in the shrinkage and temperature values take place e.g. at a second stage commencing at an age τ (which also marks the end of the first stage), the time variations in $\sigma 1$ and $\sigma 4$ (ie. $\sigma_1(t)-\sigma_{1\tau}$ and $\sigma_4(t)-\sigma_{4\tau}$ see Figure 1) can be derived in a similar process as shown above. To obtain $\sigma_1(t)-\sigma_{1\tau}$ and $\sigma_4(t)-\sigma_{4\tau}$ which replace $\sigma 1$ and $\sigma 4$ in Eqs.8,9 it is only necessary to replace the various variables in these two equations by corresponding modified values ie.

variable in Eqs. 8, 9 :	n	X	E_o	ϵ_{sn}	ΔT
replaced by :	n_τ	X_τ	E_τ	$\epsilon_{sn}-\epsilon_{s\tau}$	$\Delta T \pm \Delta T_\tau$

The subscript (τ) above is used to denote either changes in the parameter values initiated at age τ or known parameter values at end of the first stage of analysis. In a likewise manner the above derivation and modification processes can be used to obtain the time varying stresses and strains (i.e. using Eqs. [8],[9] and [6][7]) involving multi-stage changes in shrinkage and temperature values.

TYPICAL PRELOAD PERFORMANCE OF REINFORCED CONCRETE BEAMS

The results of several typical calculations are given in Figs. 2,3 to illustrate numerically the effects of shrinkage and temperature on the stress and deformational behaviour of reinforced concrete beams and slabs. The time dependent stresses and strains in concrete due to shrinkage are as shown in Fig. 2. The parameters used to obtain this set of data are p=0.02; r=0, 0.5, 1.0; β=0.9; δ=0.1, Eo=25 x 10^3 N/sq.mm; ϕn=3.0; ϵsn=0.003; X=0.67. In the calculations, creep and relaxation of concrete actually alleviate the effects of shrinkage. The effect of compressive reinforcing steel assists greatly in the redistributions of stresses and strains in the sections. These can be seen in the set of figures i.e. in between σ_1 and σ_4 as well as ϵ1 and ϵ4.

The parameters used to obtain Fig. 3 are basically the same as in Fig. 2 except that a temperature difference of 30°C is imposed in addition to the shrinkage effects in the single stage analysis and in the two-stage analysis from τ=0.6ϕ (corresponding to a time t=0.44 T) onward such temperature variation has been increased to an accumulated value = 40°C. Results of the single stage analysis is shown in the set of curves marked with (S) whereas those for the two-stage analysis is shown in the set of curves marked with (D). In the second stage of the two-stage analysis the ageing coefficient used i.e. X = 0.79 instead of 0.67. The effects of X can be seen in Fig.3.

UNDESIRABLE PERFORMANCE AT PRELOAD STAGE

The modulus of elasticity used in the above analysis corresponds to a concrete grade strength of only 20 N/sq.mm. and the free long term shrinkage value (ϵ_{sn}) is assumed 0.0003 which is considered moderate. It can be seen in Fig. 2 that the long term value of σ_4 corresponding to r=0 may well achieve 1.2 N/mm² in tension. Had ϵ_{sn} been assigned higher values, σ_4 values could had been higher than the tensile strength of concrete. The result as shown in Fig. 3 illustrate how an additional variation in temperature of 30°C to 40°C can actually cause undesirable tensile stresses and cracking (concrete cracks when its actual tensile stress exceeds its tensile strength in the set of reinforced concrete members. The tensile strength for the concrete used in this set of analysis is assumed 2 N/sqmm in numerical value.

A 30°C to 40°C temperature variation seems relatively high. However it can occur in mass concrete pour when a combined variation is caused by the hydration temperature and the ambient temperature.

PRESTRESSING ANALOGY

The developments of stresses and strains in reinforced concrete in the preload stage can also be visualized in a slightly different way. For each beam analysed the effects of shrinkage and temperature can be considered to be replaced by a natural prestressing force (P) and maintaining at certain eccentricity from the centroid of the section. The magnitude and the eccentricity of this prestressing force of nature will eventually produce the same long term (ie. at time ∞) stresses and strains in the concrete. If the section is uncracked the magnitude and eccentricity of this prestressing force (ie. Peff and e) can be obtained by equating σ_1, σ_4 obtained from Eqs.[8][9] with the expressions, $\sigma_1 = -P_{eff}/A + P_{eff}e/Z_t$ and $\sigma_4 = -P_{eff}/A - P_{eff}e/Z_b$.

To ensure the section remains uncrack the critical values of external influence from shrinkage and temperature must be evaluated. The long term value of σ_4 in Equation 9 (and similar expression in the multi-stage approach) must not exceed f_t which is the tensile strength of the concrete. By setting the long term σ_4 values to f_t and by estimating one of the two influencing variables (ie. between ϵ_{sn} and

ΔT) the limiting value of the other variable can be computed. For example, by equating σ_4 at $\phi=\phi_n=3$ to $f_t=-2$ N/sq mm in the set of numerical evaluations used in the single stage analysis (with the results as shown in Fig.3) and assuming ϵ_{sn} is maintained at 0,0003, the limiting temperature variations (ie.ΔT) for the flexural sections with r=0, 0.5 and 1.0 are respectively 14.5°C, 22.25°C and 27.37°C.

With σ_4 being assigned the value of f_t, the corresponding σ_1 can be obtained from the ratio σ_1/σ_4 obtained from Eqs. 8,9. Thus σ_1 equals 0.82, -0.92 and -2 N/sqmm corresponding to beam sections containing r=0, 0.5 and 1.0 and subject the combined external influence of respectively $\epsilon_{sn}=0.0003$, $\Delta T=14.5$°C; $\epsilon_{sn}=0.0003$, $\Delta T=22.25$°C and $\epsilon_{sn}=0.0003$ and $\Delta T=27.37$°C.

The prestressing effects producing the respective σ_1 and σ_4 values in each member can then be computed according to the above prestressing equations. In a rectangular beam of 200 mm x 500 mm (width x total depth) the following results are obtained,

r	r=0	0.5	1.0
P_{eff}(KN)	159	146	200
e(mm)	199	30.8	0

The initial values of P (ie. P_{ini}) which produce P_{eff} (in the long term) can thus be estimated on basis of the materials deformational properties. Reversing the direction of the applications of P_{ini} in these members can effectively provide counterbalancing towards undesirable preload structural performance in the reinforced concrete beams.

CONCLUDING REMARKS

The analysis with details given in this article allows the stresses, strains and deflections of reinforced concrete flexural members to be evaluated under preload, temperature and shrinkage effects. The evaluated long term stresses, strains and warping deflections can be compared to those permissible values to assess whether the effect of preload influence has already caused undesirable serviceability problems to the proposed structures. In a reverse process, by restricting the maximum tensile stress in the concrete (to value equals to its tensile strength) the limiting range of combined influence of shrinkage and temperature corresponding to a condition of serviceability limit of cracking in the preload stage can also be determined. This allows preventive measures to be taken (such as providing curing and in the control of temperature) to alleviate the anticipated structural performance. The prestressing analogy enables a reproduction of the long term concrete stresses induced by the external influence of temperature and shrinkage. Analogous prestressing effects in opposite sign can therefore be applied to the members to counterbalance (or even defuse) the undesirable preload stresses.

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Designing Reinforced Concrete Floors for Serviceability

Dimensionnement des planchers en fonction de l'aptitude au service

Bemessung von Stahlbetondecken auf Gebrauchstauglichkeit

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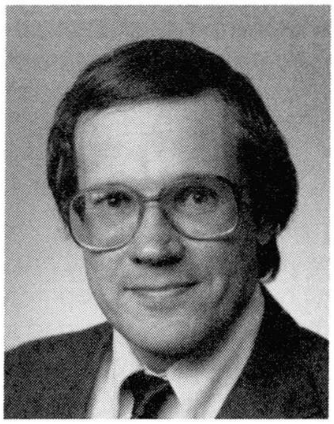

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SUMMARY

Serviceability performance of reinforced concrete floor systems must be incorporated into the design of the structure under certain parameters to avoid excessive long-term deflections. When such floor systems are subject to sustained loads, are non-continuous, or contain long spans, the design and detailing concepts must provide for structural serviceability to ensure adequate long-term performance.

RESUME

En vue d'éviter des flèches excessives à long terme dans une structure porteuse, il faut intégrer sous certaines conditions l'aptitude au service dans le dimensionnement des systèmes de planchers en béton armé. Tel est le cas lorsqu'il s'agit de structures à forte proportion de charges permanentes, à éléments uniquement isostatiques ou à longues portées; le calcul et la conception des détails doivent alors prendre en compte l'aptitude au service de ces systèmes, en vue d'assurer un comportement adéquat à long terme.

ZUSAMMENFASSUNG

In die Bemessung eines Tragwerkes muss unter bestimmten Umständen die Gebrauchstauglichkeit von Stahlbeton-Deckensystemen miteinbezogen werden. Dies ist der Fall bei hohem Dauerlastanteil, fehlender Durchlaufwirkung oder grossen Spannweiten. Bemessung und konstruktive Durchgestaltung müssen dann auf die Gebrauchstauglichkeit Rücksicht nehmen, um ein angemessenes Langzeitverhalten zu erzielen.

1. INTRODUCTION

Most reinforced concrete floor systems perform adequately. Such structures are usually designed for stress and then may be checked for serviceability. For most concrete floors systems, this procedure produces an adequate design. When reinforced concrete floor systems are subjected to moderate to heavy sustained loadings, contain long-spans, or are non-continuous, the serviceability design becomes a much more important factor in the adequate performance of the system. Under these criteria, when the serviceability portion of the design is overlooked and members are sized only on stress, excessive long-term deflection will occur. These excessive long-term deflections will adversely affect the performance of the structure. It is paramount that members be designed and detailed for serviceability to minimize excessive long-term deflections. Serviceability design should be given first priority under such criteria.

2. DISCUSSIONS

When analyzing and detailing reinforced concrete floor systems, special attention must be applied in certain portions of the structure to ensure adequate serviceability performance. From numerous investigations of existing, reinforced concrete floor systems exhibiting serviceability problems, certain parallels can be made with regard to sustained loadings, long-span members or lack of continuity in members, and excessive long-term deflections. The designer must be aware of these relationships so the serviceability requirements of the structure can be achieved. The additional cost for such design is minimal.

2.1 Sustained Loadings

The sustained, long-term loadings in some cases are obvious, such as in warehouses or libraries. In these cases, deflections are important. However, deflections are even more critical when the sustained loading is masonry partitions. In a normal classroom building, the partition weight can be substantial. Such partitions are very susceptible to cracking caused by deflections. Hence, the allowable deflection criteria becomes even stricter, with 10-mm maximum deflection or less required to avoid masonry cracking.

Sustained loadings affect the long-term, serviceability performance of a structure. Long-term, creep deflections can be two to three times the magnitude of the instantaneous deflections and will occur over several years.

Two case studies will be presented where excessive deflections were caused by sustained loadings. Under such circumstances, people occupying the building become concerned with the safety of the structure.

2.1.1 Case Study One

The first case study involves an addition to a library building. The original structure was built in 1961 with a 305-mm, flat plate slab having a minimum of three continuous spans in both directions. The reinforced, flat plate system has performed exceptionally supporting library shelving without noticeable deflections. In 1971, an addition with the same span lengths and superimposed loads was built, connecting to the original building. The slab was two-span continuous in one direction and contained more than three-span continuous in the other. The slab depth was reduced to 254 mm. No continuous top reinforcing was provided in either design. By 1977, the deflection in the addition's floors supporting the library stack shelving was an average of 50 mm. Additional deflection readings taken in 1980 showed an increase in deflections to an average of 65 mm. This additional deflection was due in part to an increase in load as the library collection expanded increasing the superimposed load. The additional load increased deflections and also contributed to an increase in the long-term, creep deflection. People occupying the building were concerned with the safety of the floors. Our stress analysis confirmed the slab capacity of the addition was adequate to support the imposed loading. The deflection analysis verified that the actual deflections were within 10 to 15 percent of the theoretical calculated deflections.

The main cause of the difference in performance of the two systems (original building vs. the addition) was the reduction of the slab thickness from 305 mm to 254 mm. The thinner slab did meet the American Concrete Institute's minimum slab-depth-to-span ratio, but actual deflections of the slab were not checked as part of the original design. No continuous, top, compression reinforcing was placed in the slab to minimize long-term deflection.

As part of our analysis, deflection calculations of the original 305-mm slab were compared to the 254-mm slab. The effective moment of interia for the original slab was 3 1/2 times greater, even though the difference in depth was only 51 mm. The increased loadings over time contributed to the continual deflections. Due to an anticipated increase in loadings, it was recommend that the building's use be changed.

A second minor contributing factor in the performance of the floor systems was the reduction from more than three continuous spans to only two spans. However, if the original slab thickness had been maintained, the deflections in the addition would still have been larger than the original building because of the two-span condition, but they would have been within acceptable limits.

2.1.2 Case Study Two

The second case study involves a college classroom building containing masonry partitions. The floor system consists of a 205-mm, reinforced concrete, flat plate spanning 7.3 m to reinforced concrete columns. Slab deflections average 64 mm causing substantial cracking of the masonry partitions. Our structural analysis of the existing conditions indicated that the slab was adequate to support the superimposed loadings, and that the excessive deflections were attributed to a lack of adequate stiffness in the structural system for the sustained, masonry partition loading. The deflection analysis confirmed the actual deflections were within 10 percent of the theoretical calculated deflection. The slab thickness did meet the American Concrete Institute's minimum slab-depth-to-span ratio. No continuous, top reinforcing was placed in the slab to minimize long-term, creep deflections.

In this case, the slab deflections were monitored over a period of several years. The use of the building remained the same with no increase in the sustained loadings. At the end of the monitoring period, slab deflections had subsided; so the masonry partitions could be repaired. It was recommended that the use of the building not be changed and no additional load be added or else the deflections would continue.

The deflection criteria for a slab carrying masonry partitions must be more stringent than a slab with no partitions.

2.2 Long Spans

Long spans can affect serviceability performance and must be designed with great care. What determines a long span is the system chosen for the area. But no matter what system is used and how light the loading is, deflections must be calculated when the depth-to- span ratio approaches the code depth-to-span limits.

2.2.1 Case Study Three

The third case study involves a college student union built in 1971. In the office space section of the structure, a 280-mm thick slab is used in a bay spanning 12.2 m by 10.9 m. Readings taken soon after the building was occupied in 1972 indicated a deflection of over 65 mm. By 1988, the deflection at midspan was in excess of 100 mm. Our analysis concluded that the slab lacked adequate stiffness to provide adequate, serviceability performance even though the slab did meet the American Concrete Institute's minimum thickness-to-span ratio.

There was no substantial, superimposed, live load and only minimal dead load of several metal stud partitions. The excessive deflections were caused by the long span. After monitoring the slab for several year, the ongoing deflection were continuing. The area was reinforced with steel beams and columns.

The use of a flat plate slab to span this area was the wrong choice of framing. A system of greater depth was required to adequately span this area. Our analysis indicated that either a waffle slab or a one-way, pan joist system would have limited the deflection to an acceptable magnitude. The building space could have accommodated such a system in this area.

2.3 Continuity in Structures

Continuity in a concrete floor system can enhance serviceability performance. If a system is designed for continuity, but if the actual boundary conditions do not provide continuity, serviceability performance will decrease.

2.3.1 Case Study Four

The fourth case study involves a three-story building containing a 205-mm thick, reinforced concrete, flat slab supported by reinforced concrete beams. In one direction, the slab spans three bays with each bay 7.9 m in length. In the other direction, there are two spans one of 7.9 m and the other of only 2.5 m. The slab deflections were greater than 50 mm. Our analysis indicated that the slab was designed as a continuous structure in both directions. But in reality, continuity was present in only one direction because the very short slab in the adjacent span did not provide full continuity. The slab did meet the American Concrete Institute's minimum thickness- to-span ratio, but the lack of continuity contributed to the excessive deflections.

After monitoring the structure for several years, it was determined that deflections had stopped. No corrective action was required. But it was recommended that no additional load be placed on the slab and the use not be changed.

The original design should have accounted for the lack of continuity in the one direction. The designer should ensure that the way the structure acts and his assumptions are compatible. Making the wrong assumption will invalidate all of the correct calculations that are made. In this case, the designer should have designed for a simple span; but he detailed for continuity. This would have minimized the excessive deflection problems encountered.

2.4 Corner and End Bays

Care must be taken when designing corner and end bays in continuous structures. Larger deflections can be expected in these areas if the span and loading are consistent with the rest of the structure. In Case Studies One and Two, 10 to 25 percent increased deflections were measured. In both case studies, the same depth framing system and spans were used as the rest of the building.

3. RECOMMENDATIONS

Do not use the code's minimum depth-to-span ratio nor stress analysis only to size members. Members must be sized on serviceability criteria. The designer must check the critical areas for initial and long-term deflections. The serviceability criteria may also be different depending on the use of the structure. A ln/360, live load, long-term deflection for a warehouse floor may be acceptable; where as in a floor with the same spans supporting masonry walls, it would not be acceptable. The long-term deflection would more likely be limited to ln/600 with a 10-mm limit.

When designing corner and end bays, try to use shorter spans and stiffer members. Under the same loading criteria, the deflection in these areas will be greater than the remainder of the floor. Deflections in these areas must be accounted for and should not be overlooked, even if the rest of the structure is adequate.

Use the most appropriate structural system. Do not use the same system used in the rest of building where in a critical area it would be marginal. The additional cost of changing the framing will be out weighted by the better performance of the structure.

Detail members for continuity; and in critical areas, try to use members with a minimum of three-span continuous. Where short spans are adjacent to long spans, design the longer spans to carry a greater portion of the load then by a continuous analysis; but still detail for continuity.

Detail the members for serviceability. Provide continuous, compression reinforcing to minimize long-term, creep deflections. Camber members for full, long-term, dead load deflections. Use members with sufficient depth to minimize deflections.

Specify construction practices that will minimize early loading of the structure. Use higher strength concrete to achieve design strength within 7 to 14 days. Specify additional floors to remain reshored, keeping a minimum of four floors to support the weight of the floor being placed. Deflections during construction will affect the performance of the structure where flat floors are required.

4. CONCLUSIONS

Under certain parameters, serviceability design of reinforced concrete floor systems becomes the most important aspect of design. If neglected, problems of excessive deflections will occur. Using the code's minimum depth-to-span ratio will not always result in a structure that will give adequate, serviceability performance. Deflections must be calculated on actual conditions of loading and continuity to ensure adequate long-term performance.

Detailing a structure for serviceability can also minimize problems. Providing adequate camber and continuous compression reinforcing will help prevent long-term, creep deflections from occurring. Construction practices are also very important in serviceability performance. Concrete strength must be achieved before construction loads are imposed on the system.

Questions of structural adequacy are raised when concrete floor systems do not meet expected serviceability performance. In some case, the lack of serviceability performance can affect use. The designer must be aware of the performance requirements for structural serviceability so that an adequate system is provided and allows the owner the flexibility to change the use of the structure if so desired.

Serviceability Analysis of Structures Including Creep Effects

Aptitude au service de bâtiments, tenant compte du fluage

Gebrauchstauglichkeit unter Berücksichtigung von Kriecherscheinungen

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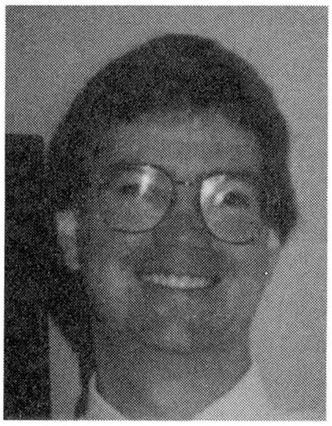

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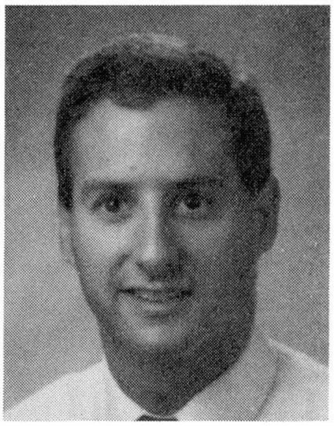

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SUMMARY

Certain current and possible future building materials (e.g. wood, concrete, structural plastics, etc.) experience time-dependent deflections when subjected to sustained load. The incorporation of the viscoelastic response of materials into the reliability analysis of structures and the effect of viscoelastic material behaviour on the serviceability performance and reliability of structural elements are discussed.

RESUME

Certains matériaux de construction présents et futurs (bois, béton, plastiques structurels, etc.) souffrent de déformations qui dépendent du temps lorqu'ils sont soumis à des charges continues. L'incorporation du comportement viscoélastique des matériaux à la sécurité de l'analyse des bâtiments et l'effet de propriétés du matériau viscoélastique à la performance de fonctionnement et à la sécurité des éléments structurels sont discutés.

ZUSAMMENFASSUNG

Manche Baustoffe, die derzeitig oder möglicherweise in Zukunft verwendet werden (z.B. Holz, Beton, Kunststoffe etc.) unterliegen zeitabhängigen Verformungen, wenn eine ständige Last auf sie einwirkt. Dieser Artikel befasst sich zum einen damit, wie viskoelastische Reaktionen von Baustoffen in die entsprechenden Zuverlässigkeitsanalysen eingearbeitet werden können, zum anderen, wie das viskoelastische Verhalten dieser Baustoffe die Funktionstüchtigkeit und Zuverlässigkeit von Tragelementen beeinflusst.

1.0 INTRODUCTION

Certain current and possible future building materials (e.g., wood, concrete, structural plastics, etc.) experience time-dependent deflections when subjected to sustained load. Depending on the material and as illustrated in Fig. 1 for structural lumber, loads of duration as short as one day may result in an appreciable increase in deflection [5]. Current deflection serviceability design checks are based on an elastic analysis and an essentially time-independent approach. Typically, the calculated elastic deflection resulting from a maximum assumed service load is multiplied by a creep factor to account for creep effects (e.g., [1, 7]). This approach does not address previous load history or even duration of the load under consideration. Owing to advances in time-dependent reliability analysis, the availability of large-scale computing for simulation, and stochastic creep models developed for construction materials, the effects of creep can be included in reliability analyses of structural building members and systems.

2.0 VISCOELASTIC MATERIALS

The characteristics of a viscoelastic material include (1) elastic deformation, (2) primary (decelerating) creep, (3) secondary (steady-state) creep, (4) zero stress creep recovery, and (5) stress relaxation. Various models are available for predicting the stress-strain-time relationship for a material, including empirical, semi-empirical, and phenomenological models. A commonly used phenomenological model which, with a single expression, predicts the five characteristics listed above is the four-element Burger model consisting of a Maxwell element added serially to a Kelvin element. The relationship between stress and strain prescribed by the Burger model is as follows:

$$\sigma(t) = K_e \varepsilon_e(t) + \mu_v \frac{d\varepsilon_v(t)}{dt} + \mu_k \frac{d\varepsilon_k(t)}{dt} + K_k \varepsilon_k(t) \tag{1}$$

where $\sigma(t)$ is the stress in the material, K_e is the Maxwell element spring parameter, μ_v is the Maxwell element viscous damper parameter, K_k is the Kelvin element elastic spring parameter, and μ_k is the Kelvin element viscous parameter. The strains $\varepsilon_e(t)$, $\varepsilon_v(t)$, and $\varepsilon_k(t)$ are those portions of the total strain owing to the Maxwell element elastic spring (elastic strain), the Maxwell element viscous damper (visco-plastic strain), and the Kelvin element (visco-elastic strain), respectively.

Solving (1) for the case of constant applied stress (i.e., $\sigma(t) = \sigma$) yields

$$\varepsilon(t) = \frac{\sigma}{K_e} + \frac{\sigma t}{\mu_v} + \frac{\sigma}{K_k}\left[1 - \exp\left(-\frac{K_k t}{\mu_k}\right)\right] \tag{2}$$

where $\varepsilon(t)$ is the total, time-dependent strain at time t beyond the application of the constant stress σ.

Depending on the material under consideration, the characteristic viscoelastic responses may differ; that is, the relative contribution of elastic, viscoelastic, and visco-plastic strain to the total strain may vary depending on the material. Regardless, the apparent time-dependent stiffness of the material, $K(t)$, may be expressed in terms of its elastic stiffness, K_e, or

$$K(t) = \frac{K_e}{\kappa(t)} \tag{3}$$

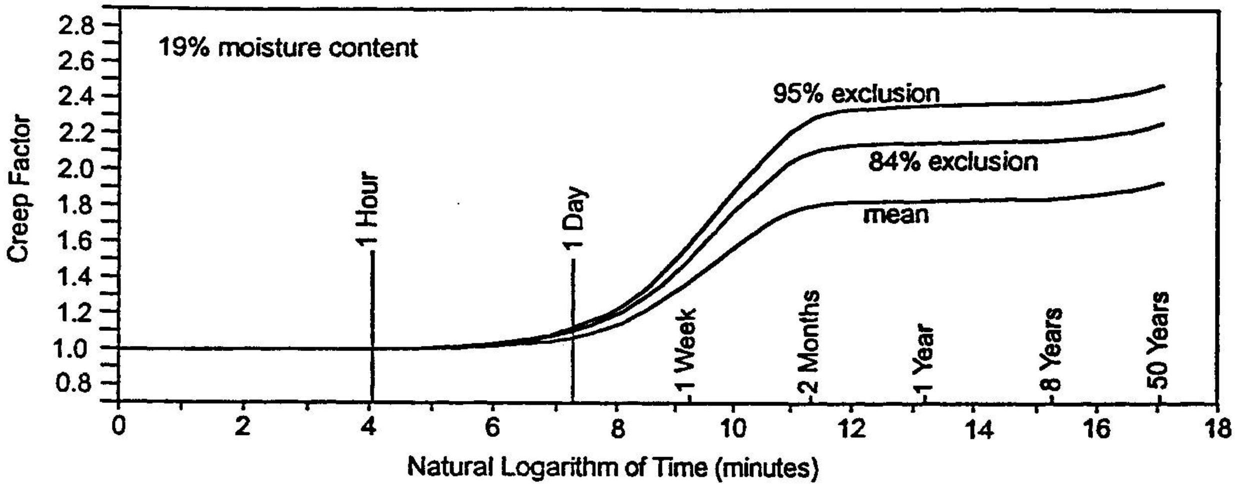

Fig. 1. Relative Creep Response of Structural Lumber [5].

where $K(t)$ is often referred to as the relaxation modulus and $\kappa(t)$ is a time-dependent creep factor. The creep factor $\kappa(t)$ is defined by the particular viscoelastic constitutive model assumed in the analysis and is a function of the load pulse form and duration. For example, using the Burger model with a constant applied stress, $\kappa(t)$ is derived from (2) as

$$\kappa(t) = 1 + \frac{K_e t}{\mu_v} + \frac{K_e}{K_k}\left[1 - \exp\left(-\frac{K_k t}{\mu_k}\right)\right] \tag{4}$$

The general relationship between the creep factor and the duration of a constant applied load for structural lumber is illustrated in Fig. 1. The four Burger model parameters used to develop Fig. 1 were assumed to be independent random variables following lognormal distributions with mean of COV values as defined in Table 1.

Table 1. Creep Model Paramter Distributions for Structural Lumber (from [5]).

	15% MC		19% MC		28% MC	
Parameter	Mean	COV	Mean	COV	Mean	COV
K_e (GPa)	11.6	0.16	10.7	0.16	9.0	0.16
K_k (GPa)	19.1	0.41	15.0	0.41	11.2	0.41
μ_k (GPa-min)	31.4	0.42	23.2	0.42	12.3	0.42
μ_v (GPa-min)	38.3	0.39	30.6	0.39	22.3	0.39

3.0 LOAD MODELS

The probabilistic modeling of loads, including dead and live loads, has been conducted by various researchers (e.g., [2, 3, 4]). For use in deflection serviceability reliability analyses including creep behavior, the stochastic load models must contain information on the load duration and arrival rate in addition to load magnitude. A pulse process model can be used to generate a complete load history for the reference period under consideration (e.g., one year, eight year, etc.). A separate load pulse process can be generated for each load type such as dead load, occupancy live load, and snow load. The generated load histories can be superimposed to allow consideration of various load combinations. In this paper, the dead plus occupancy live load combination is considered.

3.1 Dead Load Modeling

Although the self weight of a structure is well defined, the dead load is usually assumed to be underestimated by a small amount due to uncertainty in other dead load components such as permanent equipment, partitions, floor coverings, etc. [4]. Dead load statistics are summarized in Table 2.

3.2 Occupancy Live Load Modeling

Occupancy live load is typically modeled as having two components: a sustained component and an extraordinary component [2]. The sustained component includes items normally associated with the intended use of the structure. For example, furnishings and occupants in typical office and residential space are included as sustained live load. The extraordinary live component accounts for atypical use of a structure such as crowding of people during special events or the temporary use of the space for storage during renovation [3]. Statistics for both components of occupancy live load are summarized in Table 2 for two coverage areas.

Table 2. Dead and Occupancy Live Load Process Statistics.

	Intensity			Arrival	
Load Component	Mean	COV	CDF	Mean rate/year	Duration
Dead	$1.05D_n$	0.10	LN	n/a	50 years
Sustained Live					
20 sq. m	$0.24L_n$	0.90	Gamma	0.125	8 years
75 sq. m	$0.30L_n$	0.60	Gamma	0.125	8 years
Extrodinary Live					
20 sq. m	$0.16L_n$	0.90	Gamma	1	1 week
75 sq. m	$0.19L_n$	0.60	Gamma	1	1 week

4.0 LIMIT STATE FUNCTION INCLUDING CREEP

A limit state function can be derived to include creep effects from a specific design equation. Considering a deflection serviceability check where the actual deflection of a member, δ_{actual}, must be less than some allowable value, $\delta_{allowable}$, the limit state function including creep effects will take the form

$$g(x) = \delta_{allowable} - \delta_{actual} \tag{5}$$

The actual deflection is typically assumed as the elastic deflection. For a viscoelastic material, however, the actual deflection must include the elastic *and* the creep deflection components. Using (3), the actual deflection of a viscoelastic member can be written as

$$\delta_{actual} = \kappa(t) \cdot \delta_e \tag{6}$$

where δ_e is the elastic deflection. For a load pulse process, both $\kappa(t)$ and δ_e vary with each pulse. In a reliability analysis, the maximum product of $\kappa(t)$ and δ_e which occurs during serviceability reference period, or

$$\delta_{actual} = \max\{\kappa(t) \cdot \delta_e\} \tag{7}$$

must be determined and compared with the allowable deflection. The elastic deflection is a function of the actual modulus of elasticity, actual loading, and the creep factor, each of which is assumed to be a random variable, and the design span and beam moment of inertia, both of which are assumed to be deterministic.

By setting the nominal beam deflection equal to the deflection limit, the allowable deflection can be written as a function of the nominal loading and the nominal elastic modulus. By substituting this information into (5) and reducing, the following limit state equation is obtained for the deflection limit state including creep effects:

$$g(x) = \frac{E}{\phi_E E_n}\left(\frac{\gamma_D}{L_n / D_n} + \gamma_L\right) - \max\left\{\kappa_i \cdot \left(\frac{D+L}{L_n}\right)\right\} \tag{8}$$

in which ϕ_E is the resistance factor, E and E_n are the random and nominal elastic moduli, γ_D and γ_L are dead and live load factors, D and L are the random dead and live loads, D_n and L_n are the nominal dead and live loads, and κ_i is the creep factor for each load i in the reference period. Note that (8) is not dependent on the assumed allowable deflection, beam support, or loading conditions.

Typically, serviceability analyses utilize unfactored loads, i.e., $\gamma_D = \gamma_L = 1$; however, some design specifications and building codes suggest other load combinations when considering creep. For example, the *National Design Specification of Wood Construction* [7] implies a $1.5D_n + L_n$ load combination be used when considering creep effects in wood structures comprised of seasoned lumber by recommending deflections owing to "permament" loads be increased by a factor of 50%.

5.0 RELIABILITY ANALYSIS

Reliability analyses including creep effects must consider the in-time behavior of the structure throughout the specified reference period. Therefore, first-order second-moment techniques are not applicable and Monte Carlo simulation procedures must be utilized. For serviceability, reduced references periods, such as one year and eight years, are often considered in the reliability analysis. Obviously, the longer the reference period, the lower the associated reliability index. A target reliability index also must be established in order to propose appropriate creep factors for use in design. A value of $\beta = 2$ has been identified in the past as an appropriate index for serviceability (e.g., [6]). This value is generally lower than the target reliability for a strength analysis owing to the reduced consequence of failure.

Using the general viscoelastic material response presented in Table 1 and Fig. 1 for structural lumber, the load statistics presented in Table 2, and a load combination of $D_n + L_n$, the relationship between the reliability index, β, and the resistance factor, ϕ_E, was determined and is presented in Fig. 2 for one and eight year reference periods. From this example, a resistance factor, ϕ_E, of 0.45 is required to attain the target reliability index for a one year reference period and 0.35 for an eight year reference period.

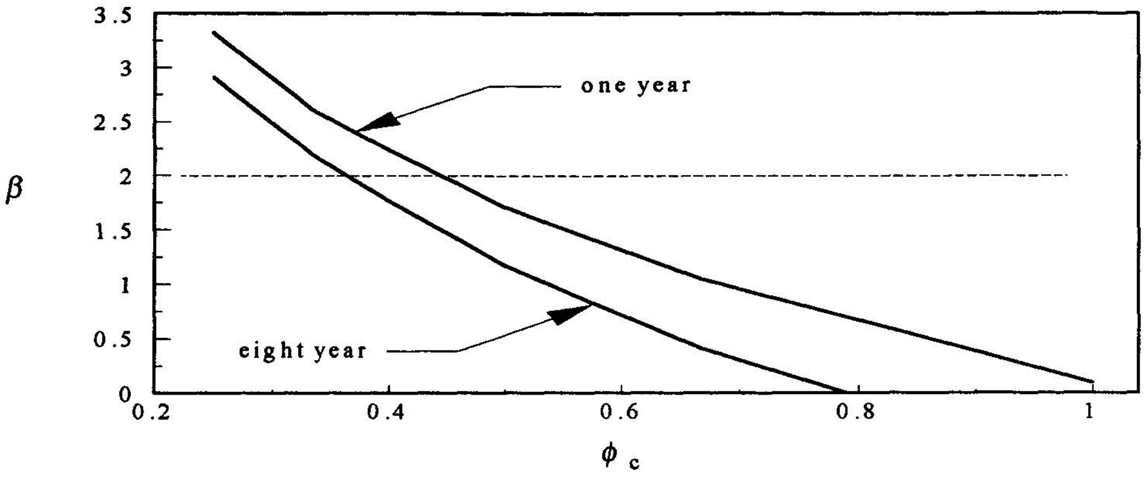

Fig. 2. $\beta - \phi_c$ Relationship for One and Eight Year Reference Periods

SUMMARY

The incorporation of the viscoelastic response of materials into the serviceability reliability analysis of structures and the effect of viscoelastic material behavior on the serviceability performance and reliability of members has been discussed. To include creep effects, the viscoelastic response of the material must be defined, and information on the load duration and arrival rate as well as load magnitude is required. The deflection limit state function was written as a function of a resistance factor, the random and nominal elastic moduli, the random dead and live loads, the nominal dead and live loads, and a creep factor for each load pulse in the reference period. The viscoelastic response of structural wood members was used to illustrate the serviceability analysis of structures including creep effects.

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Estimation of Across-Wind Response of Tall Buildings

Evaluation du comportement au vent de bâtiments élevés

Bewertung von Gegenwindreaktionen hoher Gebäude

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SUMMARY

Across-wind responses of tall buildings were investigated on the viewpoint of carrying out an estimation using simple formula in Canadian Code. The shape effects of the various cross sections on the wind responses were made clear in order to get fundamental information concerned with applicability of the formula. The across-wind responses from the formula were also compared with the data from the full-scale measurements in order to examine the accuracy of the estimation using this formula.

RESUME

L'évaluation du comportement au vent de bâtiments élevés est réalisée sur la base d'une formule simplifiée, tirée des normes canadiennes. L'effet de la forme des différentes sections sur le comportement au vent a été étudié, afin d'obtenir l'information nécessaire pour l'application de la formule. Le comportement au vent selon la formule a été comparé avec des données de mesures en vraie grandeur, afin de contrôler la précision du calcul selon cette formule.

ZUSAMMENFASSUNG

Das Verhalten von Hochhäusern gegenüber Windeinflüssen wurde anhand einer vereinfachten Formel geschätzt die aus kanadischen Baunormen hervorgeht. Die durch die Querschnittform verursachten Auswirkungen des Windes wurden nachvollzogen, um grundlegende Informationen betreffend der möglichen Anwendung der Formel zu sammeln. Die mittels Formel gerechnete Windauswirkung wurde mit effektiven, an Gebäuden vollzogenen Messungen verglichen, um die Genauigkeit der Formel zu ewrmitteln.

1. INTRODUCTION

Serviceability requirements for tall buildings have recently attracted considerable attention in structural engineering practices. Wind-induced response is the most important factor in discussing the serviceability of tall buildings.

Across-wind acceleration is usually dominant in the response caused by strong winds. Therefore, the across-wind acceleration level is used in the investigation of the serviceability. A simple formula of the across-wind acceleration, such as the formula in the National Building Code of Canada[1] and the Australian Standard[2] are generally used to ensure that the accelerations are within acceptable limits at an early stage of practical design.

In this study, the across-wind acceleration of tall buildings was investigated and discussed from the viewpoint of the estimation using the simple formula in the National Building Code of Canada. The data not only from wind tunnel experiments but also from full-scale measurement was used in this investigation.

The authors first compared between the values derived from the formula with the values based on the data from wind tunnel experiments through the use of typical tall building models that had various sectional shapes. Their aim was to discuss the shape effect of the various cross sections which affected the across-wind acceleration, and to get fundamental information concerned with the applicability of the formula. We also investigated the affect of large sections constructed at the lower part of tall buildings with regard to the use of the formula.

Secondly, we studied the actual dynamic behavior caused by strong winds using the data from full-scale measurements for a high-rise residence. Based on the records obtained from those measurements, we expressed the relation between the across-wind acceleration and the mean wind speed to get some consideration for actual response characteristics caused by winds. The accelerations from the formula were also compared with the data from the measurement to examine the accuracy of the estimation which was made through the use of formula in the National Building Code of Canada.

2. ESTIMATION OF ACROSS-WIND ACCELERATION

2.1 Wind Tunnel Experiments

Wind tunnel experiments were performed to determine the spectral characteristics of wind force components on 3D cylinders for estimating across-wind acceleration. A boundary layer wind tunnel with a working section of 3.0m x 2.5m was used for five force component measurements of the 3D cylinders

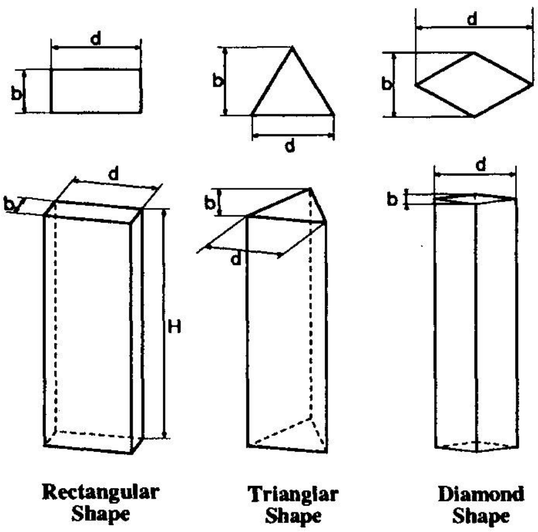

Fig.1 3D Cylinder Models

each of which had various cross sectional shapes with the same sectional area A of 0.01m^2, and the same height H of 0.4m as shown in Fig.1[3]. The shapes used were rectangular sections whose span ratio b/d (b:breadth, d:depth) was 0.33 - 3.0, triangular sections and diamond sections, each of which had the same aspect ratio H/B (B: a square root of A as a reference length of each section) of 4. Two boundary layer turbulent flows with the power law exponent of 0.20 and 0.35 for mean wind speed profile were produced for this study as shown in Table.1. A measurement of wind force was carried out through the use of a dynamic force balance which was installed at the bottom of the models. Coordinates are shown in Fig.2.

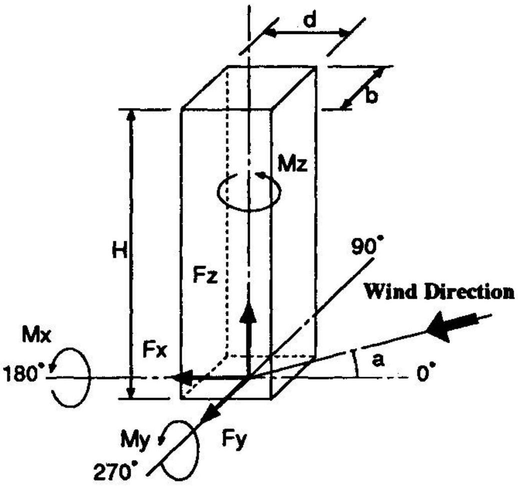

Fig.2 Coordinates

	Power Law Exponent α	Mean Wind Speed V_H (m/s)	Tubulence Intencity σ_v/V_H
Flow A	0.20	8.12	0.112
Flow B	0.35	6.92	0.156

Table.1 Characteristics of Flows

Figures 3,4,5 and 6 show the normalized power spectra of wind-induced generalized force $fS_{F1}(f)/(q_HBH)^2$ in an across-wind direction using the linear modal shape as the fundamental mode for each 3D cylinder, where q_H is the velocity pressure at the top of the cylinder, V_H is the mean wind speed at the top of the cylinder and V_H/fB is the reduced wind speed.

2.2 Across-Wind Acceleration

By using the normalized power spectra $fS_{F1}(f)/(q_HBH)^2$ obtained from the wind tunnel experiments, the r.m.s value of the response acceleration in an across-wind direction σ_a^S of a full scaled building can be calculated by following:

$$\sigma_a^S = \sqrt{\frac{\pi f_1 S_{F_1}(f_1)}{4\zeta_1 M_1^2}} \quad (\text{cm/s}^2) \ , \tag{1}$$

where f_1 is the fundamental natural frequency of the building, ζ_1 is the corresponding modal damping ratio and M_1 is the generalized modal mass of the building.

On the other hand, the r.m.s value of the across-wind acceleration σ_a^C from the formula in the National Building Code of Canada can be expressed as:

$$\sigma_a^C = \frac{2C_A}{3}\left(\frac{V_H}{f_1\sqrt{bd}}\right)^{3.3} f_1^2\sqrt{bd}\,\frac{\rho_A}{\rho_B\sqrt{\zeta_1}} \quad (\text{cm/s}^2) \ , \tag{2}$$

where V_H is the mean wind speed at the top of the building, b is the breadth, d is the depth of the building, ρ_A is the air density, ρ_B is the bulk mass of building per unit volume, and C_A is the revised factor that is pointed out by P.A.Irwin[4]. In this study, the C_A of 0.6 is used in comparisons between the response from the formula and the

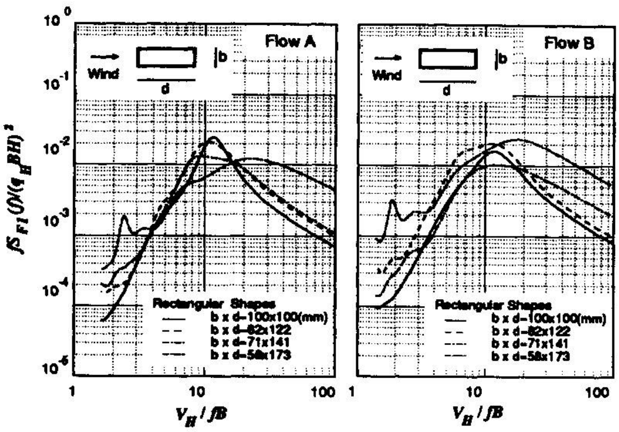

Fig.3 Normalized Power Spectra (Rectanglar Shape-A)

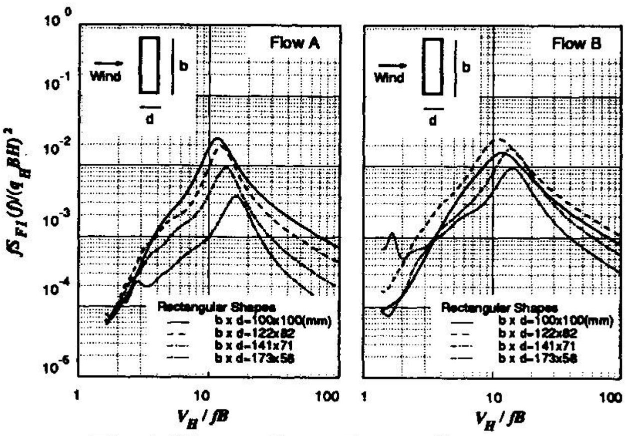

Fig.4 Normalized Power Spectra (Rectanglar Shape-B)

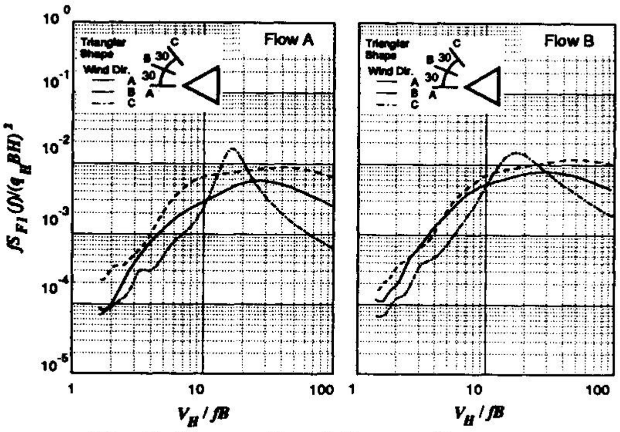

Fig.5 Normalized Power Spectra (Trianglar Shape)

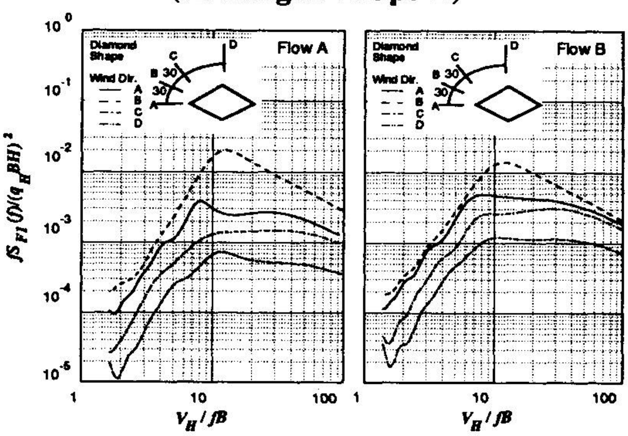

Fig.6 Normalized Power Spectra (Diamond Shape)

response based on the wind tunnel experiments.

In the investigation of the serviceability of a tall building, we must ensure that the response acceleration of the building is within the perception thresholds usually indicated as the peak acceleration value of harmonic motion. However, wind-induced accelerations of tall buildings generally are not a harmonic, but a narrow-band random process. Thus, we find the equivalent maximum acceleration corresponds to the peak acceleration of the harmonic motion by using the equivalent peak factor g_A of 2.0 to each r.m.s value from equations (1) and (2), in this study.

3. SHAPE EFFECT OF VARIOUS CROSS-SECTIONS

The relationship between the equivalent maximum acceleration and the mean wind speed V_H (> 20 m/s) at the top of the building is shown in figures 7,8,9 and 10. They show the comparisons in the equivalent maximum acceleration between the values $A^C = g_A \sigma_a{}^C$ from the revised formula in the Canadian Code and the values $A^S = g_A \sigma_a{}^S$ by the spectral modal method based on the data from wind tunnel experiments, where the r.m.s values $\sigma_a{}^S$ and $\sigma_a{}^C$ can be expressed in the equation (1) and (2), respectively. In figures 7,8,9 and 10, the characteristics of the building were

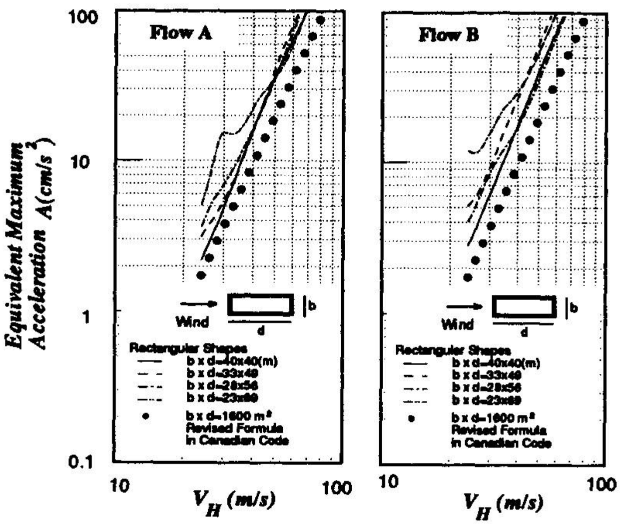

Fig.7 Equivalent Maximum Acceleration (Rectangular Shape-A)

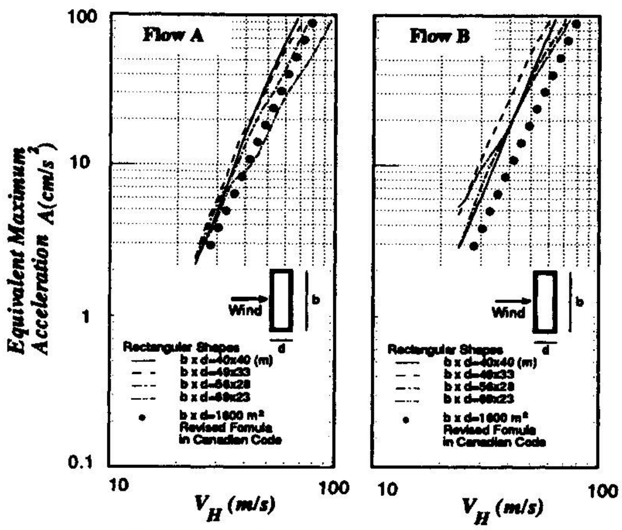

Fig.8 Equivalent Maximum Acceleration (Rectangular Shape-B)

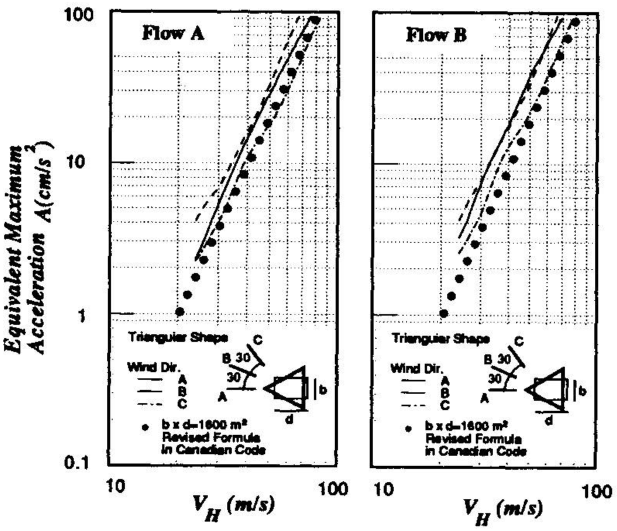

Fig.9 Equivalent Maximum Acceleration (Trianglar Shape)

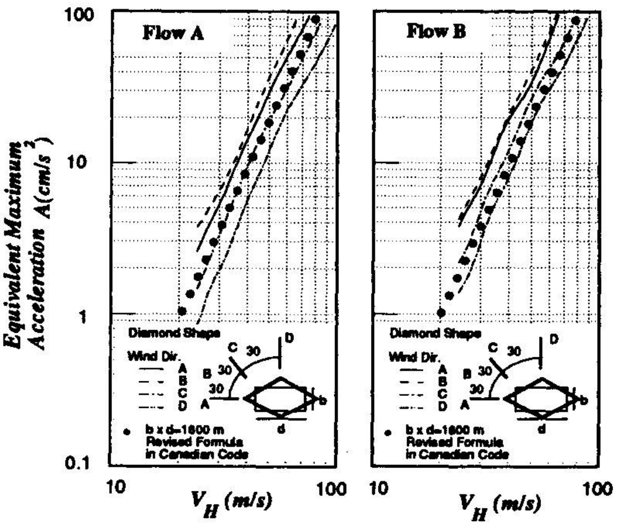

Fig.10 Equivalent Maximum Acceleration (Diamond Shape)

the following : H=160m, A (=bxd) =1600m², f_1 =0.3Hz, ζ_1 =0.01 and ρ_B =200kg/m³, for each calculation.

The accelerations A^C are less than the accelerations A^S at any wind speed V_H for each result in figures 7,8,9 and 10. The difference in both response characteristics is almost the same in the considerable range of the mean wind speed V_H. However, in the case of the rectangular section (bxd=23m x 69m), the difference between A^C and A^S becomes larger in the lower wind speed range. Thus, the formula in the Canadian Code is not suitable for the slender rectangular shape along the wind direction (for example, bxd=23m x 69m), and in turn the response characteristic from the formula differs from the characteristic by the spectral modal method based on the data from wind tunnel experiments.

From this investigation, the shape effect of the various cross sections is small in the considerable range of following the mean wind speed : 20m/s < V_H < 60m/s,

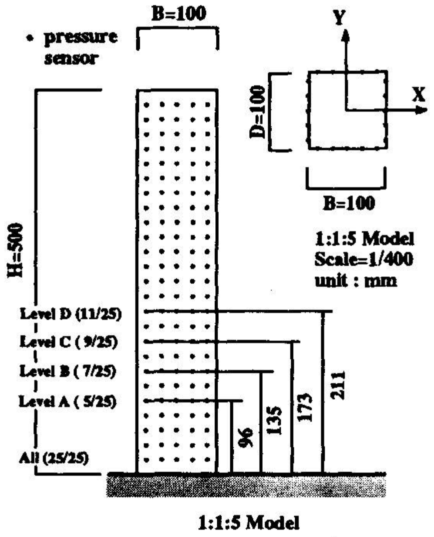

Fig.11 Presuure Model

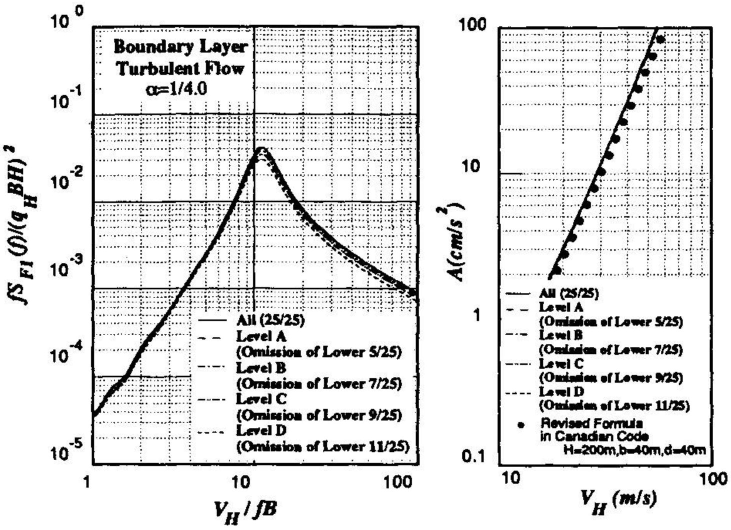

Fig.12 Effetct of the Wind Presuure Acting on the Lower Section

excluding the case of the slender rectangular shape along the wind direction. It can be also suggested that the revised factor C_A=0.6 pointed out by P.A.Irwin[4] may be small in the cross-sectional shapes that are closed with a square shape.

4. EFFECT OF LARGE LOWER SECTION

We studied the effect on the acceleration response of the large sectional area constructed at the lower part of a tall building using the pressure data from a wind tunnel experiment. The pressure model whose aspect ratio is 5 has a square cross section as shown in Fig.11. There are 25 layers of the pressure measuring section that has 5 pressure sensors in each surface of the model, making 500 pressure sensors in total. A boundary layer turbulent flow with power law exponent of 1/4 was used in the experiment.

Normalized power spectra of the generalized force $fS_{F1}(f)/(q_H BH)^2$ in an across-wind direction using the linear modal shape as the fundamental mode were calculated as shown in Fig.12. This was done under the condition that pressure measuring sections such as the lower 5, 7, 9 and 11 layers were intentionally omitted in order to study the effect of wind pressure acting on the lower part of the building. Figure 12 also shows the equivalent maximum accelerations of the building with following characteristics : H=200m, A $(=b\times d)$ =1600m^2, f_1 =0.2Hz, ζ_1 =0.01, ρ_B =200kg/m^3 and the fundamental modal shape is linear. The accelerations are calculated using the previous two methods.

From the results in Fig.12, the effect which is exerted on the accelerations by the wind pressure acting on the lower part of the building is quite small. Thus, if a large sectional area were located at the lower part of the building, the acceleration would not be influenced by the wind pressure which was distributed vertically at the lower part of the building. However, the change of the flow condition owing to the existing lower large section was not considered in this investigation.

5. COMPARISON WITH FULL-SCALE MEASUREMENT

The full-scale measurements of wind speed and response were made on the high-rise residence in Fig.13, located in the Tokyo Bay area. The building is an SRC structure with rectangular section (32m x 39m) and a height of 123.7m. From a microtremor measurement, the fundamental natural frequency is 0.43Hz, and the damping ratio of the primary mode is 1.8%. A couple of tree-cup anemometers are mounted on each side of a cross section at a level of 132m above ground. Measurements of response were carried out by accelerometers mounted at the level of 120m above ground. The records of strong winds including typhoons 9117 (Sep.14) and 9119 (Sep. 27) were obtained from the beginning of the measurements. The record of Typhoon 9119 was the strongest among them.

Figure 14 shows the mean power spectra of wind speed in Typhoon 9119. Figures 15 and 16 also show the power spectra and the time series of the response accelerations in Typhoon 9119. It was found that the acceleration in the y direction which approximated an across-wind direction was dominant and that its peak value, obtained during a 10 minutes period in which the maximum mean wind speed 23.7m/s was recorded, was 2.3cm/s². From the questionnaire investigation after Typhoon 9119, 10% of the residents who answered the questions in the higher floors felt a swaying motion, though the Typhoon passed between the time of 1:00 A.M and 3:00 A.M.

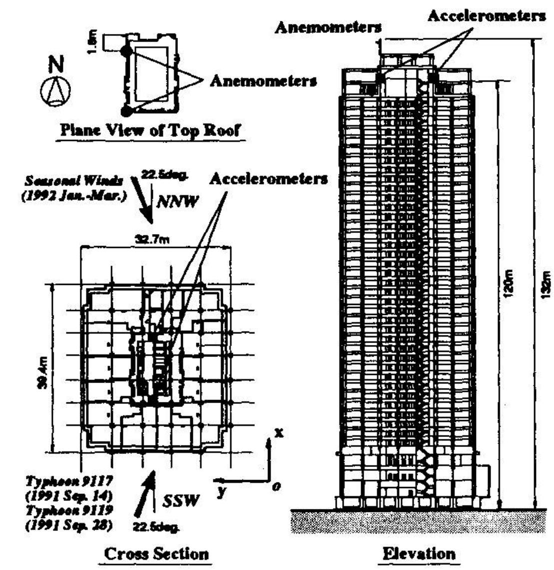

Fig.13 Cross Section and Elevation of the High-rise Residence

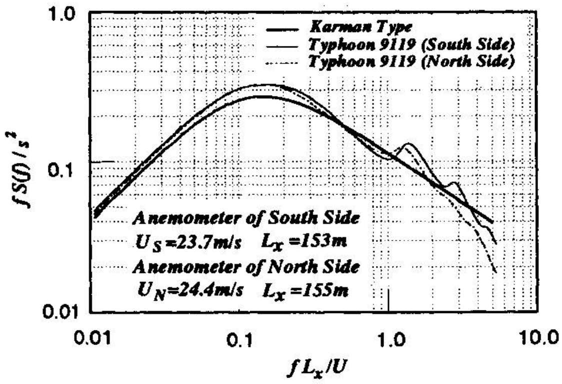

Fig.14 Power Spectra of Wind Speeds

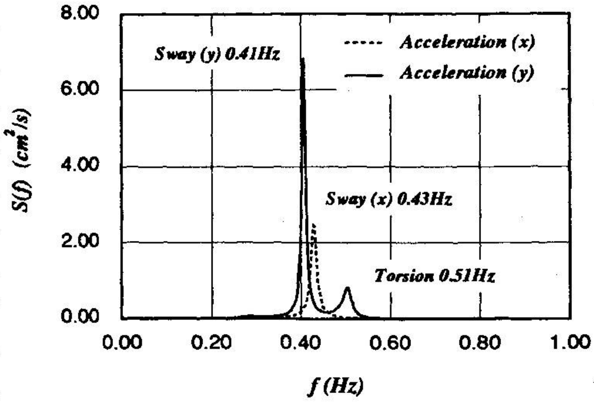

Fig.15 Power Spectra of Acceleration

Figure 17 shows the relationship between the r.m.s acceleration in an across-wind direction and the mean wind speed. The observed results in Fig.17 include the records typhoons 9117, 9119 and ordinary strong winds in the winter season. In Fig. 17, the r.m.s. accelerations from equation(2) with C_A=1.0 in the Canadian Code are also compared with measurements from the 10 minutes averaged r.m.s. accelerations.

From the result in Fig.17, the values from the formula agree with the record.

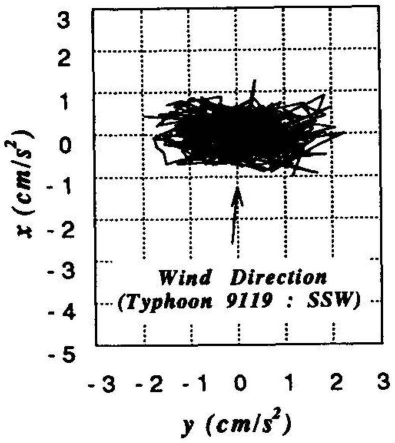

Fig.16 Time Series of Acceleration

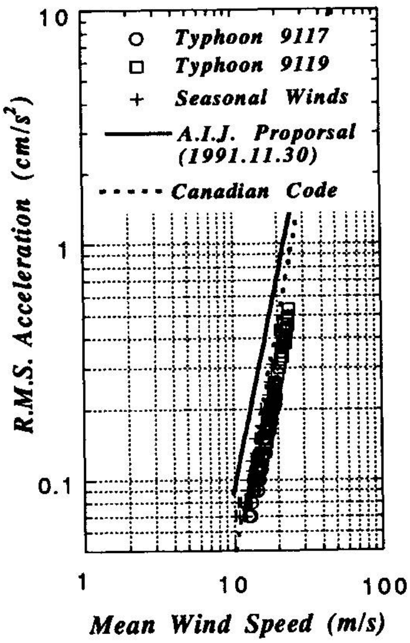

Fig.17 Comparisons with the Records from Full-Scale Measurements

5. CONCLUSIONS

The across-wind acceleration of tall buildings was investigated and discussed from the viewpoint of an estimation using the simple formula in the National Building Code of Canada. The main findings are briefly summarized as follows :

(1) The effect of shape of the various cross sections is small in the considerable range of the mean wind speed : 20m/s < V_H < 60m/s, excluding the case of the slender rectangular shape along the wind direction.

(2) It can be suggested that the revised factor C_A=0.6 pointed out by P.A.Irwin[4] may be small in comparison with wind tunnel experiment data in the cross-sectional shape that is similar to a square shape.

(3) The effect of wind pressure acting on the lower part of the building on the accelerations is quite small. Thus, if a large sectional area were located at the lower part of the building, the acceleration would not be influenced by the wind pressure which was distributed vertically at the lower part of the building.

(4) The values from the formula in the National Building Code of Canada agree with the records from the full-scale measurements.

Acknowledgments

The authors would like to express their deepest gratitude to Dr. K.Fujii and Dr. Nakamura, Wind Engineering Institute Co.,Ltd. and the members of the committee for serviceability of high-rise residence organized by the Housing and Urban Development Corp.

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Dymamic Response of Buildings on Breakwater in Monaco

Réponse dynamique de constructions sur une digue projetée à Monaco

Dynamisches Verhalten von Gebäuden auf einer Mole in Monaco

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SUMMARY

This report deals with serviceability problems in buildings founded on a planned breakwater jetty in Monaco. The problems are related to wave induced vibrations in the structure, which is heavily exposed to the Mediterranean Sea. The requirement with regard to acceleration proved to be a crucial factor in the design.

RESUME

Ce rapport traite de l'aptitude au service de bâtiments formant la superstructure d'une digue brise-lames projetée à Monaco. Les problèmes sont dûs aux vibrations résultant de la houle – la construction étant fort exposée au bord de la Méditerranée. Les caractéristiques de l'accélération se sont avérées comme très importantes pour le projet.

ZUSAMMENFASSUNG

Dieser Beitrag behandelt die Gebrauchstauglichkeit von Gebäuden, die auf einer geplanten Hafenmole in Monaco gegründet werden. Ihre exponierte Lage in der Brandung des Mittelmeers verursacht im Tragwerk durch Wellenschlag Schwingungsprobleme. Die Anforderungen bezüglich Beschleunigungen erwiesen sich als entscheidend für die Bemessung.

1. PREAMBLE

The Principality of Monaco comprises a land area of not quite 2 km² – a small urban area which, however, is characterized by expansive economic development, rendering building lots scarce and expensive. Within Monaco there are three harbours of which the main port – Port de la Condamine – is heavily exposed to the Mediterranean Sea in the sector from East/South-east to South/South-west. In order to improve the present conditions in the inner harbour the Monegasque Authorities have long been planning to construct a new breakwater jetty outside the existing moles.

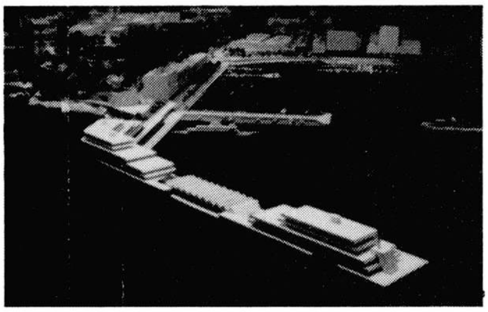


Fig. 1 Model of Port de la Condamine with new breakwater

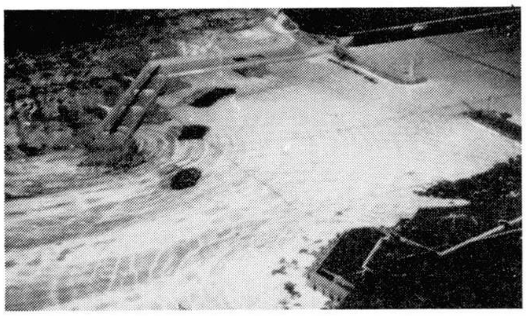


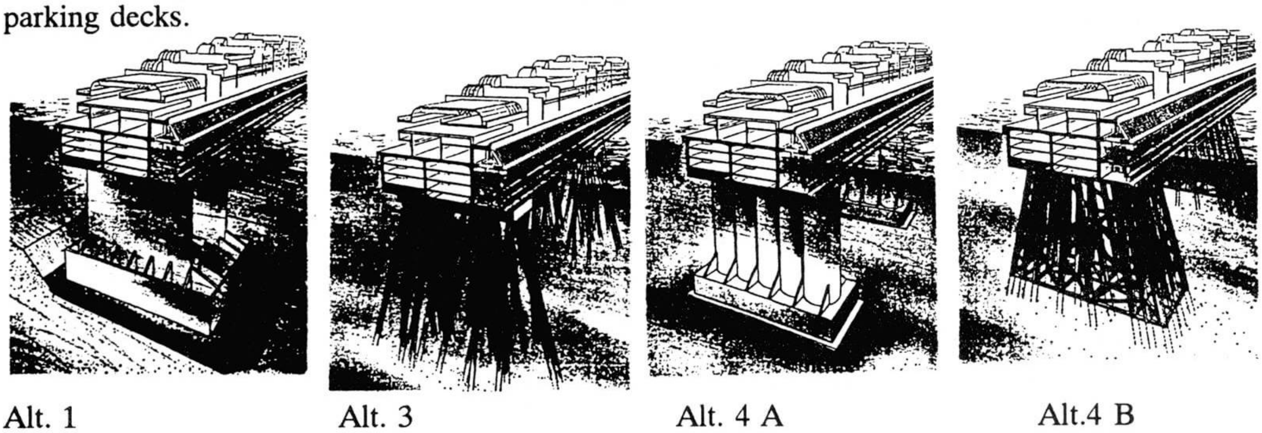
Fig. 2 Hydraulic model test

2. SCOPE OF THE PROJECT

In february 1991 Skanska Teknik AB was commissioned to carry out a preliminary engineering study in order to investigate the feasibility of such a project.

On account of the considerable depth to firm bottom (up to 60 m) it proved to be necessary to develop an unconventional design for the new jetty. The breakwater proper is planned to consist of 40–44 m wide pontoons submerged 10 m into the water. The total length of the planned breakwater is 426 m. The pontoons rest on fixed supports thus permitting some of the wave energy to pass the barrier. Four different main designs with regard to the supporting piers and their foundations were examined. (Fig. 3).

In order to promote the economic feasibility of the enterprise, the surface areas of the jetty are intended to be utilized as foundations for multi-story buildings accommodating apartments, commercial spaces and a marine terminal. The pontoons themselves accommodate three-story parking decks.



Alt. 1 Alt. 3 Alt. 4 A Alt.4 B

Fig 3 Alternative designs for the breakwater foundations

In view of the novelty of the structural concept and the open exposure to the sea, the planned breakwater belongs to the category of advanced off-shore structures. Considering the locally intense seismicity of the region and the urban development on the jetty it is clear that a number of additional complications and requirements add to those, which are normally present in off-shore engineering.
This is especially true with respect to the dynamic behaviour of the rather complex elongated construction, which may be statically described as a bent continuous beam on spring supports in all three perpendicular directions, cf fig. 5.

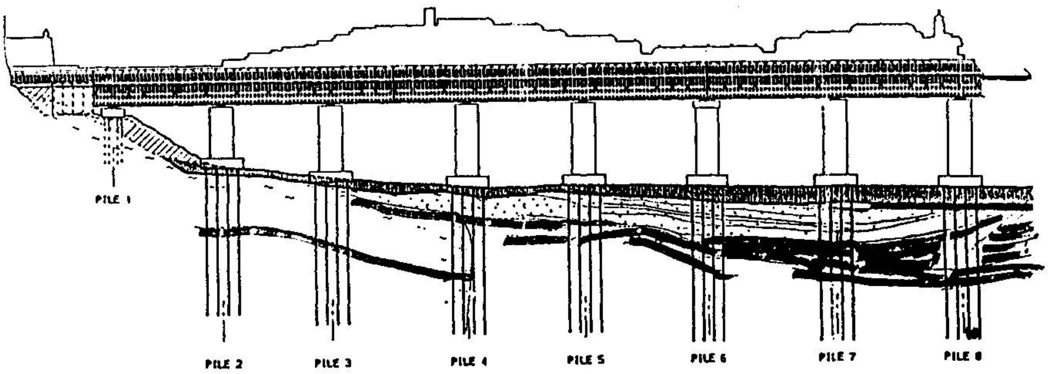


Fig. 4 Alt. 4A

In order to define the seismic effect on the foundations up to 20 modes of oscillation had to be contemplated. In the design of the various alternative solutions for the foundations, seismic action in terms of quasi-static loading proved to be governing the static design.

3. SERVICEABILITY WITH REGARD TO CONDITIONS OF COMFORT

3.1 General

The general design of the breakwater structure and the multitude of interesting issues related to it, will not be dealt with here. This paper will be restricted to problems related to the serviceability of the buildings on the breakwater with regard to dynamic response to the action of regular and irregular waves. As mentioned, four different foundation designs were studied. Although the dynamic properties varied *quantitatively* between the different designs the main features with regard to dynamic response were basically similar. Therefore data and diagrams will refer only to one of the most favourable of the alternative designs, namely alt. 3. (Fig. 3).

3.2 Dynamic properties

For the analysis of the response of both wave and seismic action it is imperative to study the dynamic behaviour of the structures. These studies were carried out with the aid of a version of the program BV STRUDL, developed by Bureau Veritas, Paris. Except for alt. 1 the integral structure was modelled in three dimensions.

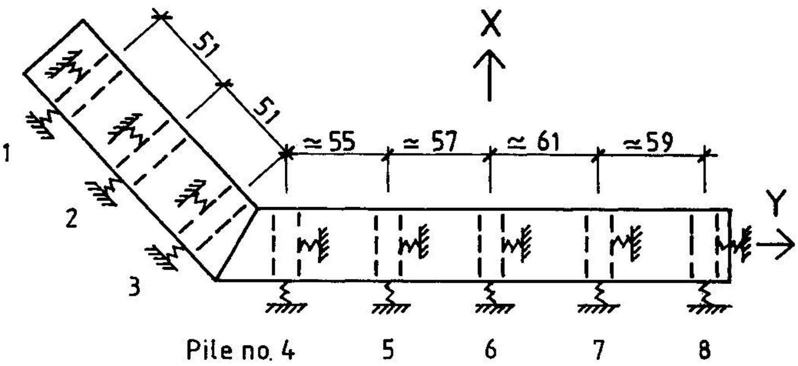


Fig. 5 Plan of model used in the analysis.
Vertical springs not shown.

Compared to a structure on dry land, the submerged structure is – from the point of view of dynamic response – encumbered by enormous added masses related to vertical and horizontal movements as well as to water contained in the caissons and the chambers for absorption of wave energy.

For the seismic analysis it was required that in each direction 90% of the total oscillating mass should be included in the analysis. This meant that up to twenty modes had to be considered. Table I gives the corresponding modes for the design alternative n° 3.

Table I Vibration modes considered in the seismic analysis for the design alternative n° 3 .

Mode	*Direction*	*Natural period T sec*	*Modal participating mass %*	Σ %
1	y	2,05	95	$\sum y = 95$
2	x	1,79	96	$\sum x = 96$
7	z	0,47	22	
8	z	0,46	36	
9	z	0,44	8	
10	z	0,43	20	
11	z	0,42	5	$\sum z = 91$

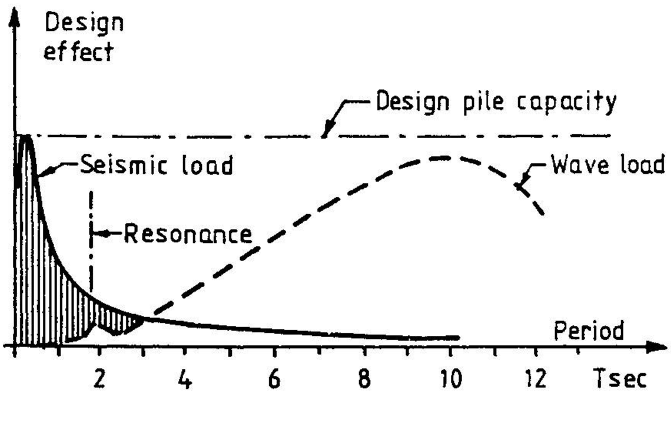


Fig. 6 Design effect on a piled foundation. (Alt. 3).

For the study of wave response, however, only the basic frequencies 1 and 2 were relevant in alt. 3. Fig. 6 shows the 'design effect' on the piles of the foundation (alt. 3) from seismic and wave loading. The diagram illustrates the importance of choosing an optimum stiffness for the integral structure. As may be seen, a fundamental natural period of e.g. 0,5 sec would have been disastrous with regard to seismicity. On the other hand a more flexible structure with a natural period of 6 to 10 sec would have been prohibitive with respect to wave loading.

3.3 Design with regard to comfort in office and living quarters

As shown in figure 6 (and table I) the fundamental mode of vibration in the x direction has a period of about 1,8 sec. This is close to optimum for the design of the foundation, the design effect of seismic and wave loading being almost the same.
There was, however, yet another important requirement to satisfy for this specific structure, namely that of human comfort in commercial areas and apartments. The clients specification prescribed that the acceleration in the most elevated apartment should not exceed 0,005 g during the 1 year storm, defined by H_{max} = 4,5 m and T = 7,0 sec for regular waves.

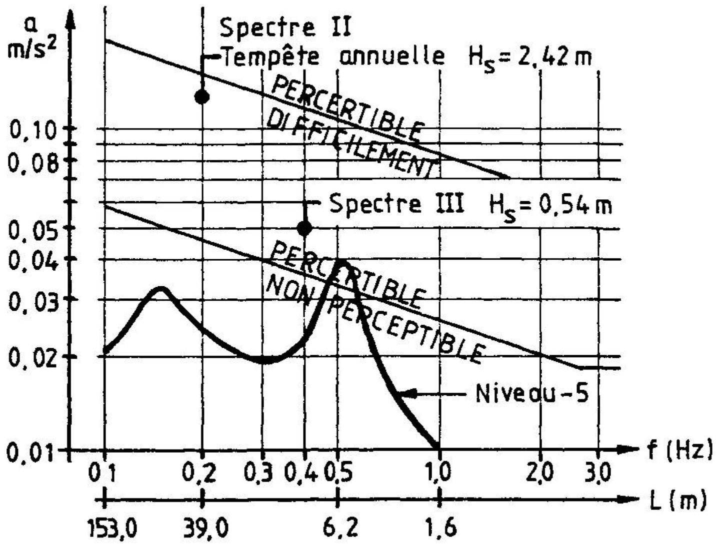


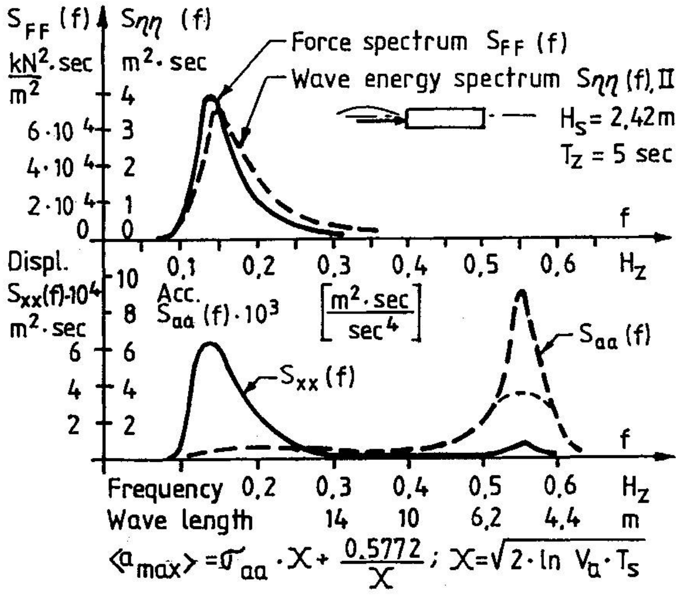
Fig. 7 Maximum acceleration for regular waves and for spectra II and III

Fig. 7 displays the relationship between regular wave frequency and maximum acceleration.
The non-exceedence criterion ($a_{max} \leq$ 0,005 g)is fulfilled in this case. The curve indicates, however, that the contribution of smaller waves in an irregular sea could possibly be important. Hence it was found necessary to perform a supplementary *spectral* analysis.
The irregular sea state was defined as a Pierson-Moskowitz spectrum with the following data:

H_{max} = 4,5 m
H_s = 2,92 m
T_z = 5 sec

Fig. 8 depicts the basic wave elevation spectrum, the corresponding force spectrum and the displacement as well as the acceleration response spectra. As may be seen from the diagram the response force spectrum, which is directly proportional to the displacement spectrum, retains the main features of the wave energy spectrum, reflecting the moderate effect of resonance on the design forces on the structure.

However, the conditions for human comfort are linked with the maximum velocity (shear forces in the building) and acceleration. It is therefore interesting to note the tremendous increase of spectral density in the acceleration spectrum near the natural frequency of the breakwater structure. Table II gives the results of spectral analysis for the breakwater structure for two wave spectra of which spectrum n° II corresponds to the sea state specified by the client with respect to comfort criteria.
Spectrum n° III represents a sea state where the maximum spectral density of the wave energy spectrum is located near the natural frequency of the structure.



As may be seen in fig. 8 the response spectrum for maximum acceleration shows a marked displacement of the peak spectral density from the frequencies of maximum wave energy spectral density to the band of resonance frequency. This implies that the contribution of smaller waves – with wave lengths in the order of 6 m – to maximum acceleration is of major importance. As indicated in fig. 7 the limit acceleration of 0,005 g is exceeded for both spectrum II and III. It is even likely that any chosen spectrum with an appreciable content of smaller waves in the resonance frequency band would generate unacceptable accelerations.

Fig. 8 Spectrum n° II, sea elevation and force
Spectrum n° II, acceleration and displacement response spectrum

Table II Maximum acceleration at level +24,2 (top story). Alternative 3, wave direction East/ South–east, spectral analysis

Spectrum	H_{max}	H_{sign}	T_z	*Displace-ment*	*Accele-ration*	*Storm duration*
	m	*m*	*sec*	*mm*	*m/sec²*	*hours*
II	4,5	2,42	5,0	32	0,120	4
IIIA	1,0	0,54	2,5	7	0,050	4

The accuracy of this prediction is, however, subject to the validity of the basic assumption that the time–force history of the irregular sea is identical and simultaneous for every section of the c:a 400 m long structure. This is probably not very true for wave lengths of 5–8 m. For this reason the finally predicted acceleration was based on a modified response spectrum where the spectral density of smaller waves had been reduced on the basis of engineering judgement.
A continued study of the serviceability of the structure with respect to vibrations and comfort would therefore have to address the probability of the simultaneousness of impact from smaller waves on a structure longer than about 50 times the crucial wave lengths.

4. CONCLUSIONS

In the design of the various alternative solutions for the foundation, seismic loading was governing the design. However, with regard to the relative feasibility of the different alternatives, spectral analysis of dynamic wave response demonstrated, that the *serviceability* requirement related to the comfort in living quarters, was a strongly decisive factor with a potential of eliminating some of the alternative designs.

Predicting Floor Response Due to Human Activity

Prédiction du comportement d'un plancher soumis au mouvement des gens

Deckenverhalten auf menschliche Bewegungen

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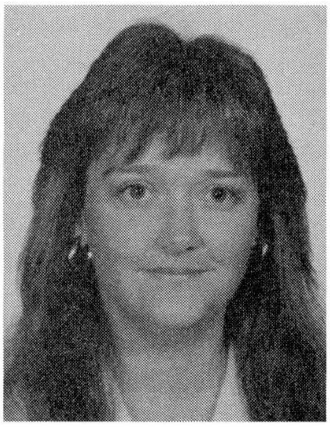

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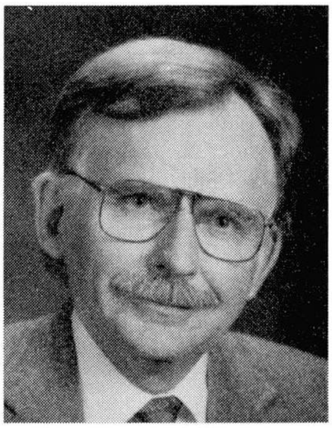

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SUMMARY

This paper presents the results of a study which compares actual floor vibration response data to data obtained from dynamic finite element models. Accelerations are reported for three actual floors subjected to various forces. The floors systems, which vary in size and complexity, are concrete and metal deck with steel supporting members consisting of open web joists and/or steel beams.

RESUME

Cet article présente les résultats d'une étude comparant la réponse à la vibration de planchers réels à celles obtenues par l'utilisation de modèles dynamiques d'éléments finis. L'article comprend un compte rendu des accélérations de trois planchers soumis à une diversité de forces. Les systèmes de planchers, dont les dimensions et le degré de complexité diffèrent, sont construits en béton armé et en acier, et ils sont supportés par des treillis métalliques légers ou des poutres en acier.

ZUSAMMENFASSUNG

Der Beitrag stellt gemessenen Deckenschwingungen Resultate von dynamischen Berechnungen mit finiten Elementen gegenüber. Beschleunigungen wurden für drei Deckensysteme mit verschiedenen Lasten ermittelt. Die Deckensysteme bestehen aus Beton auf Profilblechen, die mit Stahlleichtbauträgern und/oder Walzprofilen unterstützt sind.

1. INTRODUCTION

Providing a floor system which is free from annoying floor vibrations is most economically accomplished in the design phase of the building. This, however, requires design procedures which will predict a problem floor before it is built. Although not explicit in many of the available design procedures, three aspects must be considered to provide an acceptable floor system. The first involves determining the permissible levels of floor vibration for the intended occupancy. Second, an appropriate excitation function for the intended activities must be determined. The third aspect involves the prediction of floor response to specified excitation functions. These three aspects have, either separately or in combination, been the subject of a great deal of research.

1.1 Permissible levels of floor vibration

Permissible levels of floor vibration depend on occupancy requirements. Occupancy requirements range from limitations of sensitive equipment to the "comfort" of the occupants. Ungar, Sturz and Amick [1] present peak velocity requirements for several facility types which have either sensitive equipment or uses, such as optical research systems or microsurgery.

Comfort of the occupants is a function of human perception. This perception is affected by factors including the task or activity of the perceiver, the remoteness of the source and the movement of other objects in the surroundings. A person is distracted by acceleration levels as small as 0.5%g in an office or residential environment. People involved in an activity such as aerobics may be comfortable with acceleration levels up to 5%g [2]. Multiple use occupancies must therefore be carefully considered. Ellingwood and Tallin [3] provide guidelines for limits of steady-state vibration, damping and peak acceleration, according to occupancy.

Perception is also affected by the nature of the vibration response. Steady-state vibration will disturb at much lower levels than vibration which is transient. Many different scales are available which address the subjective evaluation of floor vibration. Factors included in these subjective evaluations include the natural frequency of the floor system, the maximum dynamic amplitude (acceleration, velocity or displacement) due to certain excitations, and the amount of damping present in the floor system.

1.2 Excitation functions for various activities

At the present time, most of the design criteria utilize either a single impact function to assess vibrations which are transient in nature or a sinusoidal function to assess steady-state vibrations from rhythmic activities. The floor excitation in office and residential environments is generally due to the intermittent movement of occupants. The vibration response is, therefore, considered transient and floor response is commonly evaluated on the basis of a single impact function. The heel-drop impact is the basis for several design criteria. For rhythmic activities such as dancing, aerobics, hand clapping, etc. the forcing function is commonly approximated as sinusoidal with a magnitude pertaining to the activity and the number of participants [4].

The use of these simplified excitation functions is driven by the computational abilities of design engineers and can be implemented using hand calculations for simplified systems. More complicated excitation functions, which require advanced or automated analysis procedures, are available. In particular, Ellingwood and Tallin [3] quantify a force function due to a single person walking. The force varies in time, magnitude and location. It may also be useful to note that there are entire scientific journals dedicated to the study of human locomotion, providing information which could be developed into statistically based excitation functions.

1.3 Prediction of floor response

As noted previously, many of the currently available design criteria were developed for implementation in hand calculations. These calculations vary widely in their complexity. One recommendation for commercial environments [3] uses a limiting static deflection to assure an acceptable dynamic response. Other recommendations require that the first natural frequency be kept greater than the second or third multiple of the excitation function to avoid resonance[2]. This is commonly referred to as frequency tuning.

More complex criteria include methods for computing a dynamic amplitude to be included in the subjective rating. Two such examples are the methods presented by Murray [5] and Allen[2]. These methods are derived from closed form solutions for excitation functions applied to a simple beam, a cantilever beam or an equivalent single degree of freedom system. Effects of the individual components (beams, girders and columns) are then superimposed to compute a system response.

2. IMPLEMENTING NEW ANALYSIS CAPABILITIES

The currently available methods, which utilize hand calculations, for determining dynamic response are often much too limiting. Irregular bay shapes and continuous members are nearly impossible to assess with such methods. For these situations, gross simplifications or other analysis measures must be implemented. As personal computer hardware and software capabilities increase, while prices decline, the application of dynamic computer analysis solutions is becoming quite practical for the design engineer. PC based structural analysis software is becoming increasingly user friendly and computationally very powerful. Graphical and menu driven input processors make the modeling of entire floor systems simple and efficient. Dynamic capabilities allow the designer to subject a floor system to time dependent forces at any location. Modal damping can also be included in the model. Output results include response spectra and time histories for displacement, velocity and acceleration, at any node in the model.

Assuming that accurate results are obtained, this type of analysis capability could have a great impact on the determination of floor vibration serviceability requirements. The accuracy of analysis results can begin to be assessed using case studies of the dynamic responses for actual floors. This paper summarizes case studies of three floors of varying complexity. All of the floor systems are concrete and metal deck with steel framing members.

2.1 Description of the analytical models

The dynamic finite element analyses presented in this paper were carried out on a commercially available structural analysis software package. The finite element models utilize beam elements for modeling the steel framing members, and plate elements for modeling the concrete slab and deck, in a single plane. These models are best described as grid models. The level of complexity was chosen with the design engineer in mind.

Plans of the three floors are shown in Appendix A. Floors A and B are bare test floors and floor C is a finished floor in an occupied clothing and shoe retail store. Floor A consists of two deep joist members supported by masonry walls. The heavy lines indicate the locations of continuous support. The model consists of a mesh of approximately 0.61 m (2 ft) square and modal damping was estimated, from experimental data, at 2.5%. Floor B is a 9.14 m (30 ft) square bay supported continuously along two edges by a masonry wall, as shown in the plan. The finite element mesh is 0.305 m (1 ft) square and the modal damping was estimated, from experimental data, at 1%. Floor C is a multi-bay floor system. The bay analyzed is trapezoidal in shape. The model is broken up into a mesh of approximately 0.76 m (2.5 ft) square and modal damping was estimated, from experimental data, at 5%.

The framing members are steel joists (lightweight truss members designed to carry distributed loads). The floor slabs are constructed of lightweight (Floors A and B) or normal weight (Floor C) concrete on 14.3 mm (9/16 in.) metal deck and have a total thickness of 63.5 mm (2.5 in.). Floor C has rolled steel W sections for girders which are continuous over the supports. This is a common lightweight framing configuration for small to medium size commercial buildings in North America.

At relatively small dynamic loads, such as those created by people walking, the steel members behave compositely with the concrete slab; therefore, transformed section properties must be utilized to obtain an accurate response from the analytical model. Due to the nature of the connections in joist supported floor systems, joist members are modeled with pinned member ends, resulting in no moment transfer in the framing members over the center girder and no rotational restraint at the columns. However, it must be noted that this is not the case for rolled shapes with shear connectors; the rotational restraint of the shear connector is often sufficient to transfer moment at small loads.

The models presented use a 2.67 kN (600 lb.) ramp function over a time period of 50 milliseconds as the excitation force. The location of this force, and the time data, is noted as point A on each of the plans in the appendix. The actual floor is excited by the heel-drop of a 0.845 kN (190 lb.) man, which is closely approximated by the ramp function noted above.

2.2 Summary of results

Comparisons of finite element and actual readings for floors A, B, C are shown in Figures 1, 2, and 3, respectively. Graphs are included which show acceleration time histories over a period of five seconds. A fourier transform was performed on each of the signals and the frequency responses are also represented in graphs in Figure 1,2 and 3.

All three models very accurately predicted the dominant frequencies in the dynamic responses. In floors A and B the peak accelerations in the experimental data exceeded those predicted by the finite element model. This may be attributed to an inaccurate prediction of the magnitude of the actual heel drop impact. The actual heel-drop impact was not measured. The experimental and model peak accelerations noted for floor C are amazingly similar. Particularly when the complexity of the actual floor system and relative simplicity of the model used are considered. In this model, the actual heel drop impact may have been more closely predicted by the model.

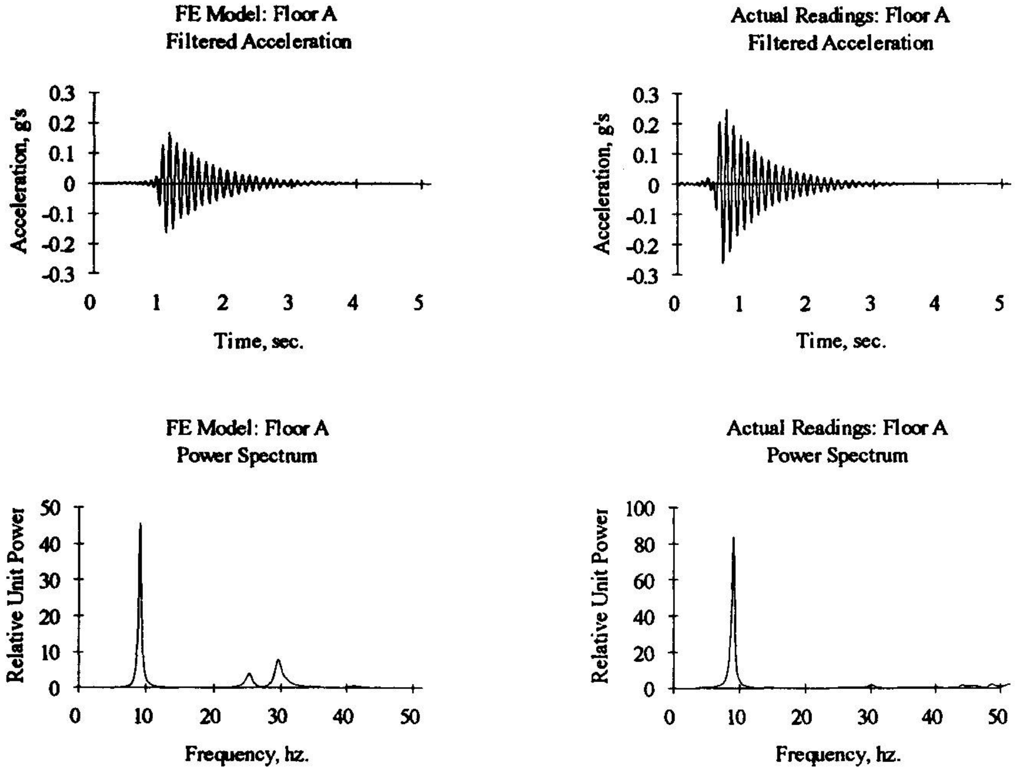

Figure 1 Comparison of Finite Element Results to Actual Results for Floor A

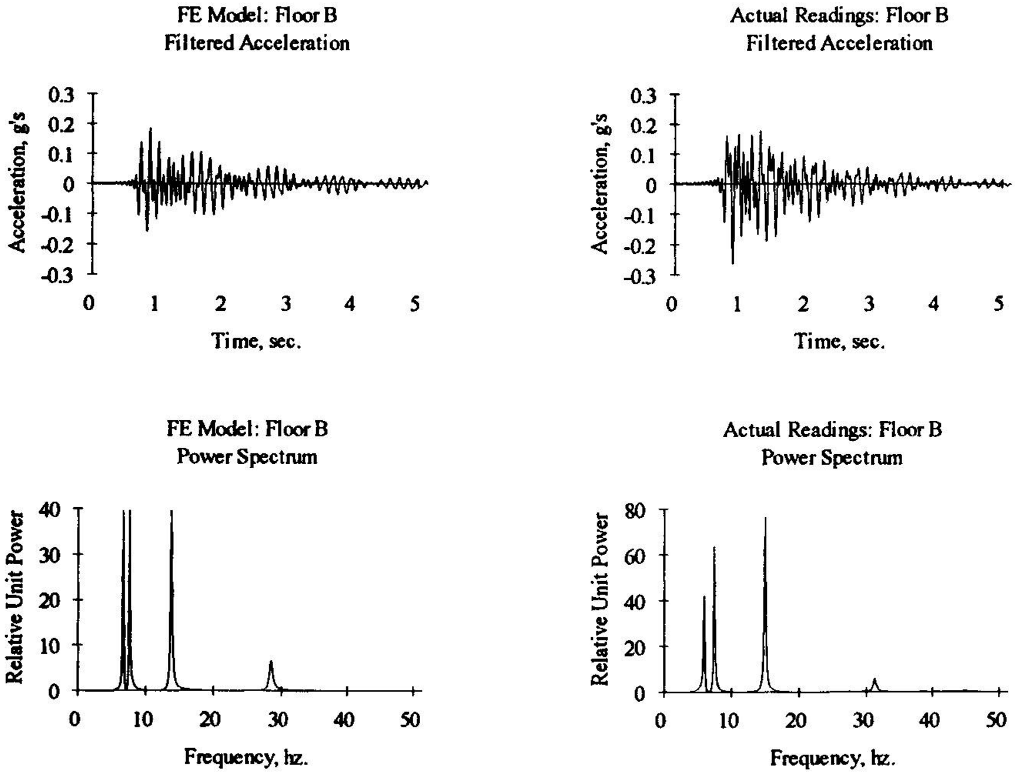

Figure 2 Comparison of Finite Element Results to Actual Results for Floor B

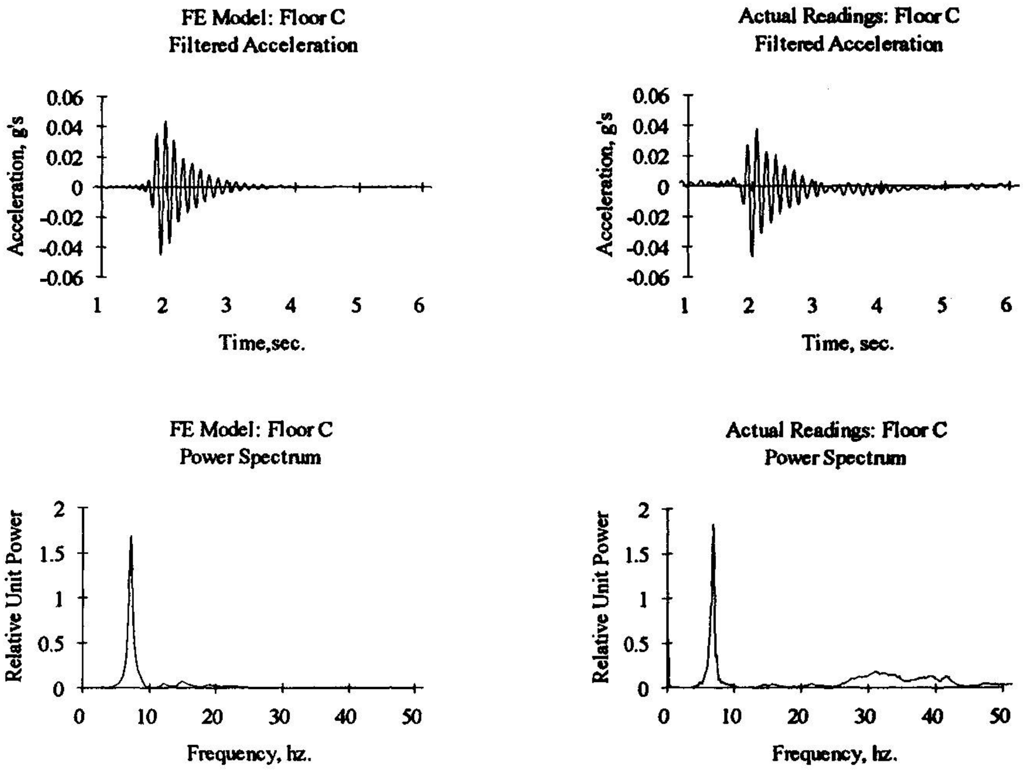

Figure 3 Comparison of Finite Element Results to Actual Results for Floor C

3. CONCLUSIONS

Most of the currently available design criteria were developed with practical computational methods in mind. With the ever increasing use of computer analyses, the definition of "practical computational methods" is becoming much more advanced. In order to fully utilize the expanding analysis capabilities, criteria must be developed which include more exact excitation functions, explicit limitations for frequency ranges and dynamic amplitudes for different occupancies, along with accurate model constraints (i.e. the level of complexity required for a model to accurately predict vibration response).

ACKNOWLEDGEMENTS

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APPENDIX A

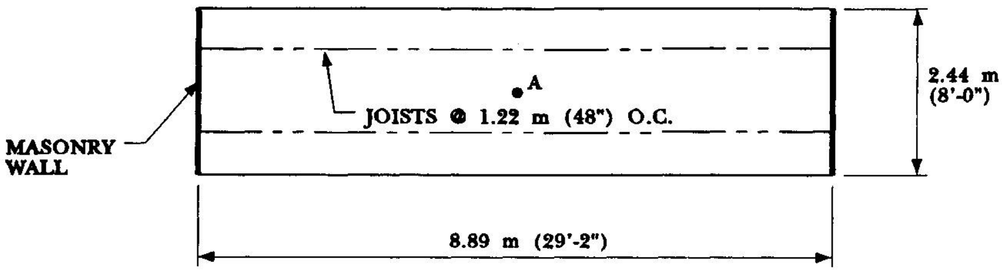

Figure A.1 Plan of Floor A

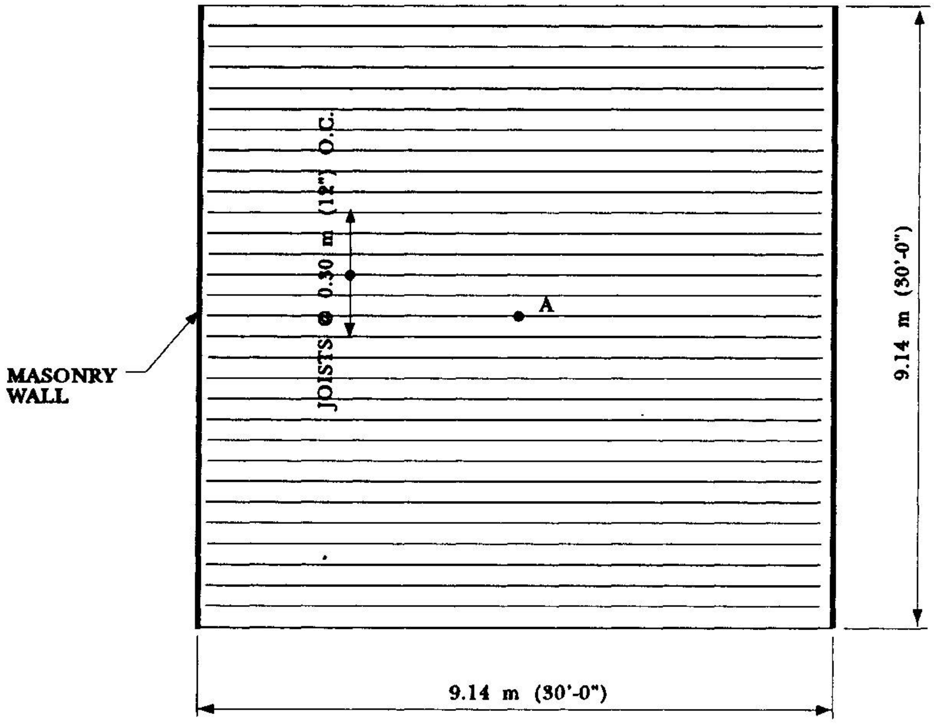

Figure A.2 Plan of Floor B

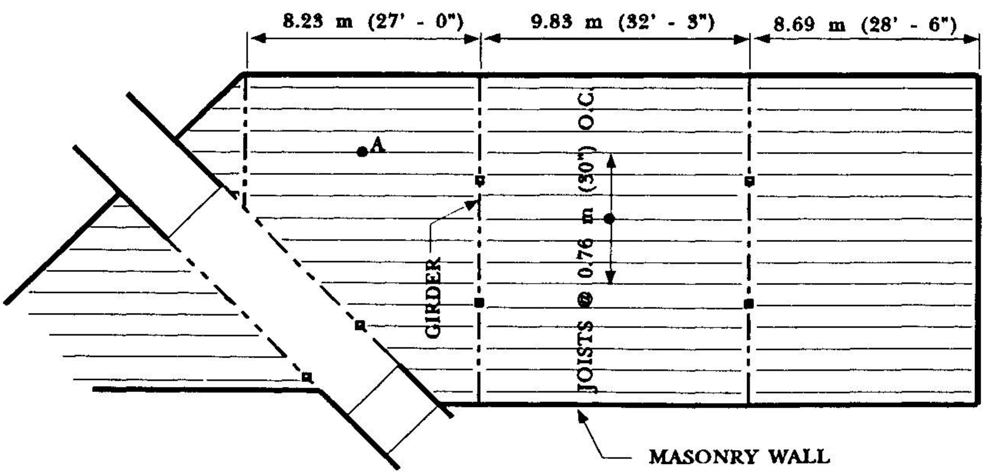

Figure A.3 Plan of Floor C

Vibration Characteristics of Shallow Floor Structures

Caractéristiques de vibration des structures de plancher mince

Schwingungseigenschaften gedrungener Deckenkonstruktionen

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SUMMARY

This paper examines the vibration behaviour of shallow floor structures constructed from precast concrete slabs supported on steel beams. A number of floor areas in three buildings were tested to determine their fundamental frequencies. The floors were in different stages of construction, and thus offered the opportunity to identify the effects of different constructional steps. Three floor areas were selected for more comprehensive forced vibration tests and also for examining their response to human actions. This paper presents the results of these tests and takes a detailed look at the test that yielded the largest response to the human loads. Finally, it discusses general serviceability issues.

RESUME

Cet exposé traite du comportement des vibrations de structures de plancher mince construites à partir de dalles en béton préfabriquées posées sur des poutres d'acier. On a effectué des essais sur un certain nombre de surfaces de plancher réparties dans trois bâtiments, afin d'en déterminer les fréquences fondamentales. Les planchers sélectionnés en étaient à différents stades de construction, ce qui a permis d'identifier les effets de chaque étape de la construction. Trois surfaces de plancher ont été choisies pour des essais plus complets de vibration à outrance ainsi que pour examiner leur réaction aux activités humaines. Les résultats ainsi que des questions d'utilité générale sont examinées.

ZUSAMMENFASSUNG

Dieses Referat untersucht das Schwingungsverhalten von gedrungenen Deckenkonstruktionen aus Fertigbetonplatten auf Stahlträgern. Eine Reihe von Deckenbereichen in drei Gebäuden wurde zur Bestimmung ihrer Grundfrequenzen geprüft. Die Decken waren in verschiedenen Baustadien begriffen und boten daher eine ausgezeichnete Gelegenheit, die Auswirkungen von verschiedenen Bauphasen zu ermitteln. Drei Deckenflächen wurden für eingehendere erzwungene Schwingungsprüfungen und für die Untersuchung ihrer Reaktion auf menschliche Einwirkungen ausgewählt. Das Referat beschreibt die Ergebnisse dieser Prüfungen und behandelt im Detail jene, bei der sich die stärkste Reaktion auf menschliche Belastung ergab. Schliesslich werden allgemeine Fragen der Gebrauchstauglichkeit besprochen.

1. INTRODUCTION

The work reported in this paper forms part of a wider investigation into the vibration serviceability limit state for floors. The basic problem being that people walking or running across floors will produce vibrations, which for some floors may prove perceptible or even annoying to other users of the building. If the annoyance is of sufficient severity it may disrupt work or suggest that the structure provides a bad work environment, hence this would be termed a serviceability failure. In the past this has not been a significant problem, but with the trend for longer-span and lightweight floors, the dynamic response of the floors grows increasingly important. It is now necessary for the engineer to consider this point in design; in fact, for some designs, the dynamic behaviour may prove to be a limiting design consideration.

The early analysis methods used by non-specialist UK. designers gave minimum design values for natural frequency. The new Eurocode 3 [1] recommends that *the lowest natural frequency of construction floor should not be lower than 3 cycles/second*, which is somewhat lower than the early recommendations. This limiting frequency criteria will be discussed later. In 1989, a design guide [2] primarily for composite floors was produced, but this needs to be broadened and simplified. At the moment there seems to be no clear consensus view of serviceability under foot fall vibration backed by reliable experimental data. The ultimate aim must be a new comprehensive but simple guide for the designer to ensure his floors are serviceable, including a clear guide on calculating natural frequencies and other limiting parameters.

The investigation reported in this paper examines the vibration behaviour of an example of shallow floor structures or slim floors as they are called in the UK. The concept of shallow floor structures in pre-cast or in-situ concrete, where the supporting steel beam is incorporated within the slab depth, arrived in the UK from Scandinavia in 1990. Early projects designed by Swedish companies incorporating the 'top hat' type of fabrication were joined by another unique Swedish concept marketed as the ThorBeam [3]. At the same time British Steel were studying alternative forms of floor construction in Scandinavia and, following research and development work, have recently launched the concept of slim floor beams built from rolled sections and plate. The benefits of shallow floor structures of this nature derive from the direct costs of reduced building heights and the absence of downstand beams in ceiling voids.

Although British Standard floor loading is 2.5 kN/m^2 it is rare for designs to be based on less than 3.5 kN/m^2 (including 1 kN/m^2 for partitions) and more usually 5 or 6 kN/m^2. The higher loads together with a static deflection criterion of span/360 under imposed load produces floor stiffnesses where dynamic behaviour is unlikely to be critical for normal spans. However, at some load level there will be a transition between the dominance of static deflection and dynamic behaviour as the design criterion.

2. DETAILS OF STRUCTURES TESTED

In July 1992 an opportunity arose to test a large number of floor areas in three buildings which were being constructed. This provided a chance to evaluate the performance of a mixed structure of steel columns, composite steel/concrete ThorBeams and pre-cast hollow core units, all of which were proprietary items arriving from different sources. In addition to measuring the finished structural response, these buildings, being in different stages of construction, offered the opportunity to identify the effects of different constructional steps which converted individual component actions into the complete structure.

The general floor grid for all three buildings was 7.2 m square. The typical floor cross-section incorporates a ThorBeam, fabricated from plate materials, which directly supports pre-cast hollow core units. The design superimposed load in the office areas was 6 kN/m^2. The office floors were 200 mm pre-cast concrete units with C35 structural concrete fill in the beam, and 75 mm topping over the pre-cast concrete units in a strip 1500 mm wide about the centre line of the ThorBeam. Reinforcement was included within the structural topping to guarantee the shear capacity and also to provide restraint into the cores of the hollow core units. Tests were also conducted on the plant rooms which were designed for 7.5kN/m^2 but the results are not presented here.

Restrictions in the length of the paper mean that the investigation cannot be reported fully, hence the paper will focus on the key results and attempt to show the changes encountered during the construction process and how they affect the vibrational behaviour.

3. TEST PROCEDURE

Two types of test were used to determine the dynamic characteristics of the floors. First an impact test, which is simple and quick, was used to establish the fundamental frequency of the floor. Then for three selected floors, forced vibration tests were used to determine all the characteristics of the fundamental modes. For the three floors tested using forced vibrations, further tests were conducted to monitor the vibrations induced by human actions. It should be noted that the floors were not expected to encounter vibration problems, but it was thought that the investigation would provide details for calculations if longer spans or lighter construction were used in the future.

3.1 Impact tests

The basic principle of an impact test, is to cause the floor to vibrate by introducing a single impact and to monitor the ensuing decay. The analysis of the decay of vibrations can provide an accurate measurement of the frequency of the response and on some occasions can be used to estimate damping, although these damping estimates need to be treated with caution as they usually overestimate the true value. For the site tests, a geophone (velocity transducer) was set up in the centre of the floor to monitor response. The floor was subject to an impact produced by one person using the 'heel drop' method. The response was recorded on a computer, and processed using an FFT procedure to produce a power spectrum which could be examined on site to yield the frequency of the response (usually the fundamental frequency of the floor).

3.2 Forced Vibration tests

In order to obtain the best quality data on the characteristics of a floor a forced vibration test is desirable. The forced vibration testing procedure is described in full in [4]. The procedure uses a vibration generator whose frequency can be accurately controlled, and by identifying, and then exciting the fundamental mode, accurate measurements of frequency, damping, stiffness and mode shape can be obtained.

3.3 Vibration from human actions

On each of the three floors subject to forced vibration tests, the response of the floors were recorded as the floor was excited by a series of human actions. The floor response was monitored using an accelerometer set up near the centre of the floor. In all there were six tests on each floor area and the same person was used for each test. These involved walking and then running across the floor, and then at the centre of the floor slow jumping, quick jumping, slow running on the spot and quick running on the spot. The response was recorded for eight seconds and the recording analysed to determine the peak response and the frequency content of the response.

4. BASIC STRUCTURAL CHARACTERISTICS,

Tests were conducted on six levels in three buildings, which included 106 floor areas, four single precast planks and four single ThorBeams.

The four simply supported planks (precast concrete units) gave frequencies ranging from 4.88 to 5.86 Hz and were very lightly damped. The grouting of the planks not only joined them together but also provided a variable connectivity with the secondary steel beams of the framework to provide some two-way spanning action. This was confirmed by taking the mode shape measurement on one area which was in this state. In one case, it was noted that there was no connectivity with the secondary beams and here the measured frequency was 4.88 Hz. Where connectivity with the secondary beams was noted, but where the ThorBeam was not concreted the frequency range was 5.37 to 7.81. Finally when the ThorBeam was concreted the frequencies of the floors ranged from 7.37 to 10.99 Hz.

For calculation purposes, the simply supported beam provides a simple model for calculating the frequency of the precast concrete units. The situation where the ThorBeams are not grouted is more like a simply supported plate, albeit the supports are not rigid, and the completed floor is akin to a plate with two simple and two clamped supports. It is interesting to note that the ratio of frequencies of simply/clamped to simply/simply supported plates is 1.47, and the ratio of the average measured values of the corresponding floors is 1.39. Also if the floor had been designed for a lower imposed load, perhaps 3.5 kN/m^2, and used (say) 150 mm hollow core units rather than 200 mm units, by taking a simple ratio of the design inertia and mass, it suggests that the frequency would be about 0.75 times that of the 200 mm floors.

The above tests results exclude the final finishes, but from experience it can be assumed that, in general terms, the changes which occur with the finishes will follow a reasonably obvious sequence. The false floors will provide increased stiffness and damping, and these increases will be less significant for longer span floors. For areas which have been stiffened by significant structural alterations, adding stairways etc., significant increases in stiffness will result, hence higher frequencies. For areas which have just had a significant increase in supported mass a decrease in frequency will occur. For intermediate cases which have some stiffening and some added mass then only small changes in frequency will result. There will be, however, a general trend for all the damping values to increase, albeit some by only a small amount.

5. MEASUREMENT OF RESPONSE

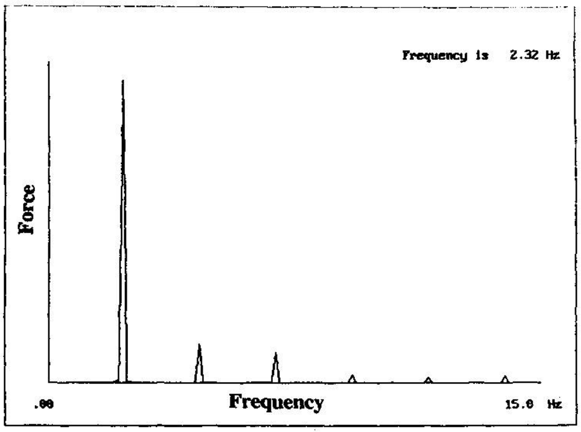
Three floors were subjected to forced vibration tests. Two of the floors were finished and on the other floor (B7) the concrete had not been cast on the ThorBeam. The unfinished floor had a fundamental frequency of 7.21 Hz and damping of 1.44% critical. Of the finished floors C2 had a frequency of 7.92 Hz and 1.69% damping and B2 had two closely spaced modes at 8.23 and 8.71 Hz with approximate damping values of 1.99% and 0.75%. The mode shape measurements on all the floors showed two-way spanning behaviour within the floor area and significant excitation of adjacent floor areas.

The three floor areas were used to obtain measurements of their response to human actions. The peak accelerations are presented in the following table with a note of the dominant frequency of the monitored response. It should be noted that these frequencies are not necessarily the fundamental frequency of the floor. In a number of cases, significant accelerations occurred at several other frequencies besides the dominant one, and for these cases the secondary frequency is given in brackets.

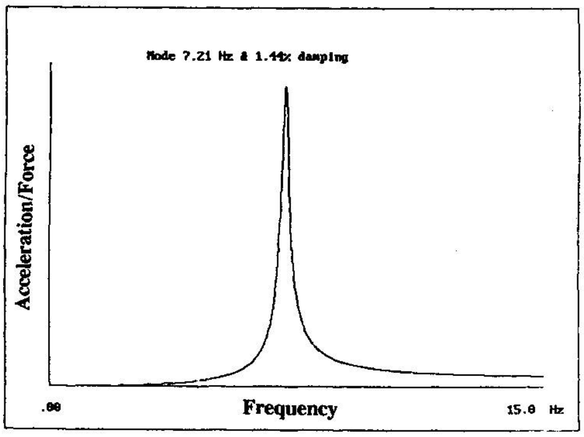
DESCRIPTION	C2 peak accel. g	dominant freq. Hz	B7 peak accel. g	dominant freq. Hz	B2 peak accel. g	dominant freq. Hz
Walking across floor	0.0210	7.93	0.0162	7.81	0.0078	7.69 (8.30)
Running across floor	0.0251	9.77 (8.06)	0.0350	7.20 (7.81)	0.0247	8.79 (6.35)
Jumping on spot (slow)	0.0582	8.18 (7.81)	0.0827	7.08	0.0432	8.79 (7.93)
Jumping (quick)	0.0814	7.57 (9.89)	0.1330	6.96	0.0389	8.06 (5.37)
Running on spot (slow)	0.0234	8.06 (7.81)	0.0190	7.20 (7.45)	0.0123	8.54 (8.06)
Running (quick)	-	-	0.0395	4.52 (7.20)	0.0274	9.79 (4.76)

Examination of the event which produced the biggest accelerations is instructive. The recorded acceleration time history looks remarkably like a single frequency response and indeed the autospectrum shows that the response is almost solely at 6.96 Hz. This is not the resonance frequency of the floor, nor is it the jumping frequency, because there is a upper limit of approximately 3.5 Hz at which people can jump. A closer look at the autospectrum showed another smaller peak at 4.64 Hz and another much smaller peak at 2.32 Hz. What is being observed is responses at multiples of the jump frequency at 2.32 Hz. Examination of the force time history from someone jumping, shows that it is like a series of half sine waves with zero force when the jumper leaves the ground. The frequency content of this time history shows a large force at the jumping frequency and significant but reducing forces at whole multiple values of the jump frequency.

The three adjacent figures illustrate what is happening. The upper figure is a spectrum of the force generated when one person jumps at 2.32 Hz (with a contact ratio of 0.67), and the forces at multiple values of the jumping frequency can be seen. (The frequency resolution of this spectrum is relatively crude but has been selected for compatibility with the lower figure.) The centre figure shows an acceleration transfer function for a one degree of freedom model with a frequency of 7.21 Hz and damping of 1.44% to correspond to the values measured on B7. The structural acceleration is calculated by multiplying the forcing function by the transfer function to obtain a result like that shown in the lower figure. However, the lower figure is the actual spectrum measured on B7 for the response to the quick jumping. At 6.96 Hz only one third of the full resonance amplification is obtained, so if the jumping had been quickened to 2.403Hz then the response could have trebled. This shows the importance of avoiding resonance, and is the reason why a fundamental frequency of more that three times a dancing frequency is often mentioned for dance floors and gymnasia.

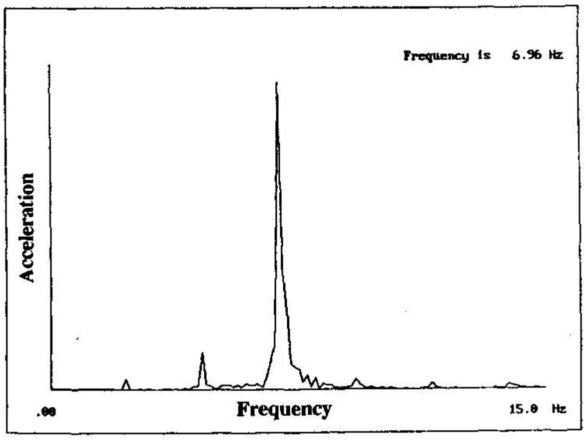


Calculated forces for jumping at 2.32 Hz



Acceleration transfer function

The situation for normal serviceability requirements is somewhat different to the dance loads because the normal loads are from walking and running, not jumping. For walking the frequency of pacing is generally between 1.5 and 2.5 Hz and for running this range may extended up to 3.5 Hz; and, as with the jumping, energy can be input at whole number multiples of the basic frequency. The old design recommendations of a minimum floor frequency of 5 to 6Hz, were effectively avoiding the chance of resonance from the energy input at the second multiple of the walking frequency, albeit they were probably just set to be above the walking/running frequency. The EC3 recommendation of a minimum of 3 Hz, could lead to floors where resonance could occur, and therefore produce problems.



Measured Response on B7

Damping is of importance as it controls amplification at the resonance, the amplification being inversely proportional to the damping. It also controls the rate of decay of vibrations following any excitation and this will influence a users perception of the floor acceptability. As it is not possible to calculate damping values, values obtained from tests on similar structures should be used if required.

6. SERVICEABILITY CONSIDERATIONS

There are several national and international guides which give acceptable levels of vibration and the acceptability is influenced greatly by the duration of the vibration. There are also many other factors which are important, but for the purposes of this paper, a simple/rough indication of thresholds is required. The table produced below is extracted from a CEB guide [5], which is stated to be in broad agreement with the values found by many research workers as an indication of human perceptibility thresholds for vertical harmonic vibrations for a person standing.

Description	Freq. range 1-10 Hz peak accn. mm/s^2	Freq. range 1-10 Hz peak accn. g
just perceptible	34	0.0035
clearly perceptible	100	0.010
disturbing/unpleasant	550	0.056
intolerable	1800	0.183

If the measured responses on floor area B7 are compared with the above values it can be seen that the quick jumping on the spot would have produced vibrations of a disturbing/unpleasant nature. However, this isn't really the critical factor, because if someone was jumping at 2.32 Hz by your desk you would be able to see the cause of the vibration and hence not be concerned. The more common criteria would indicate that the running or walking would both be clearly perceptible for the bare floor, but with the addition of false floors and furnishings they are likely to fall into the just perceptible range. Hence the floors tested herein would not have a serviceability problem when completed.

The serviceability problem is technically quite straightforward, because the dynamic characteristics of any floor can be calculated, or approximated, and given a loading function the floors response can be calculated. However, there will be a range of possibilities of load functions with various forces, forcing frequencies and duration's. These will result in a range of accelerations which can be compared with some acceptance criterion. The acceptance criteria are actually quite difficult to establish and depend on many variables, including individual perception, but, as there is quite a lot of information available on the subject, this should be feasible. The easiest way of dealing with the problem, is still to try and avoid it, by designing a floor with a sufficiently high fundamental frequency.

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8. ACKNOWLEDGEMENTS

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Dynamic Response of a Composite Slab in an Office Building

Comportement dynamique d'une dalle composite d'un bâtiment commercial

Dynamische Reaktion einer Verbunddecke in einem Verwaltungsgebäude

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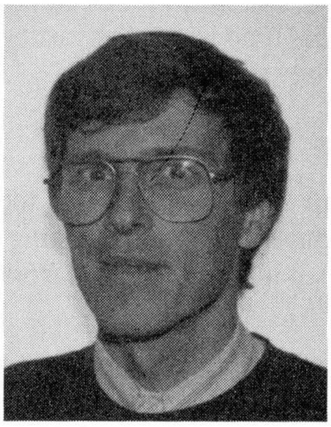

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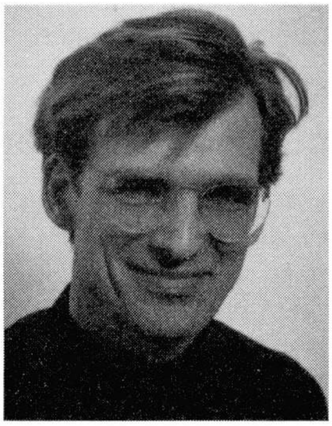

Eilif Svensson, born 1945, obtained his MSc and PhD degree at the Technical University of Denmark. Since he has been working in civil and structural engineering as a consulting engineer with particular emphasis on stability and dynamic in relation to larger steel structures and has written several papers on these issues.

SUMMARY

The structural dynamics serviceability of a composite floor of an office building is investigated using an FEM modal analysis. The results presented show that the structure behaves well when exposed to people walking and running, and acceptably when exposed to many people dancing.

RESUME

L'aptitude au service, pour des cas de charge dynamiques, d'une dalle composite d'un bâtiment commercial est examinée avec une analyse modale par éléments finis. Les résultats présentés montrent que la structure se comporte très bien pour les cas de personnes marchant ou courant. Le comportement structurel est acceptable pour le cas de personnes dansant.

ZUSAMMENFASSUNG

Das dynamische Verhalten einer Verbunddecke in einem Verwaltungsgebäude wurde mittels einer modalen Finite-Element-Berechnung untersucht. Wie die Resultate zeigen, eignet sich diese Bauweise sehr gut für gehende oder laufende Personen und ist noch annehmbar unter Tanzbelastung.

1. INTRODUCTION

During the last decades it has been a tradition to use precast concrete elements for all kinds of buildings in Denmark. We are now successfully introducing a solution based on a combination of concrete elements and steel beams (see fig. 1 & 2). Mixing materials this way we utilize the best properties of each material with regard to economy, bearing capacity, deformations and fire protection. One remaining problem to be clarified is the dynamic response of the structure exposed to human activities, such as walking, running, jumping and dancing.

In the ISO codes (ISO 2631) [1] and related national codes [2] serviceability limits for accelerations (comfort criteria) are introduced for various types of buildings and other structures.

A major challenge for the present lighter structure (see fig. 1 & 2) was the verification of its ability to meet the above mentioned serviceability requirements. To clarify this we examined the structure with simplified calculations and with a more accurate finite element (FEM) modal analysis using the PC based program ALGOR.

2. HUMAN ACTIONS ON THE FLOOR STRUCTURE

Various investigations have been carried out [3] with the goal to set up loading criteria for the dynamic human actions on structures. As a result some simplified formulas for people walking, running, jumping, dancing etc. have been established [2],[3]. An example of the dynamic load from one walking person with a relative weight 1 is shown in fig. 3.

3. SIMPLIFIED CALCULATIONS OF EIGENFREQUENCIES

The floor structure (see fig. 1) can as a first assumption be analyzed as two independent structural systems - a one-way slab and a continuous beam with 2 interior supports.

For the simply supported one way spanning slab we determine the lowest eigenfrequency using well known formulas. The higher eigenfrequencies will be influenced by the integration with the steel edge beams, loading from facades and the elastic support of the cross steel beams. Using the total dead loadings and one third of the live load we have calculated the lowest eigenfrequency to 5.17 Hz in the example shown. This is identical to the result for a traditional fully precast structure. In [3] the recommended lower limit for the eigenfrequency is to 8.0 Hz. This means that we might encounter problems using a completely traditional precast hollow core slab structure.

For the continuous cross beam we assume that the bending stiffness corresponds to the steel section and that the mass corresponds to the loadings on the beam from the slab. Using this the lowest eigenfrequency can be calculated to 3.8 Hz. If we had been using a traditionally simply supported precast T-beam, the eigenfrequency would have been 5.0 Hz. It seems clear that the flexural stiffness of the steel beam will be greater than our assumption due to the interaction with the concrete slab and that the vibrating mass will be lower than the assumed due to the flexibility of the slab. This means that the eigenfrequency will be greater than 3.8 Hz in the real structure.

From the above remarks it seems clear that a more detailed investigation of the integrated floor structure is required.

4. FEM ANALYSIS

For the further investigations we establish a FEM model of the integrated slab / beam structure supported on the stiff columns. The horizontal movements are disregarded.

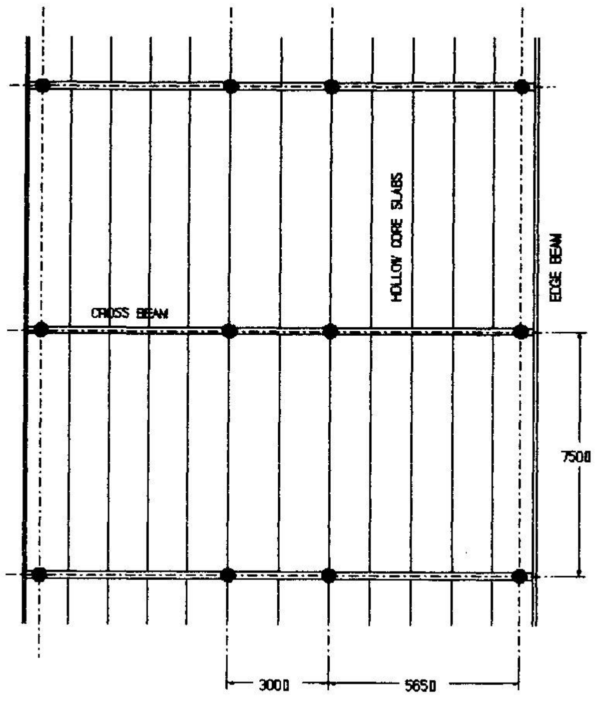

Figure 1. **Plan of a part of the floor in the office building**

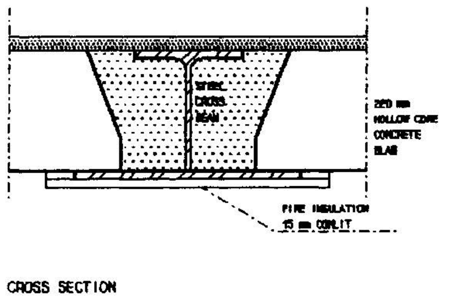

Figure 2. **Section in the cross beam.**

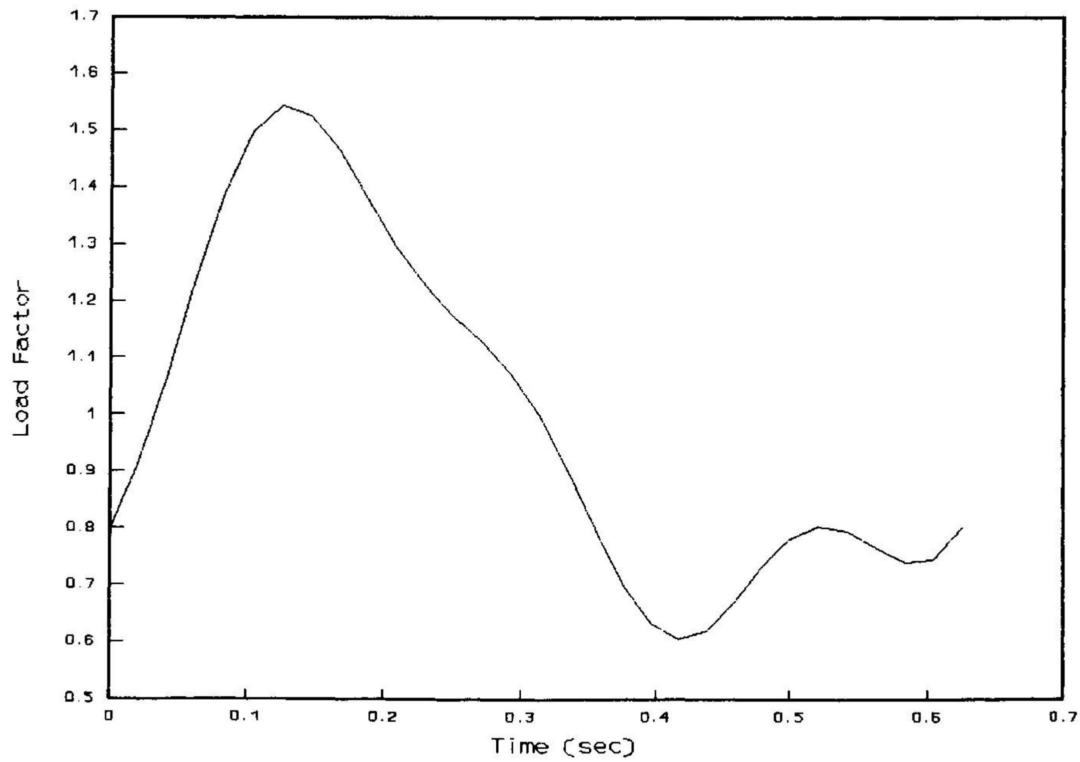

Figure 3. **Load Factor versus time for a walking person [3].**

The cross and edge beam elements are modelled very simply with their own properties and with nodes mainly identical with the nodes of the plate elements. Above the facade columns we have created extra nodes for the beam elements.

The hollow core slab elements are modelled as isotropic plate elements with 4 nodes each. Between the elements in the longitudinally direction (of the building) we model a pure shear connection to account for the shear lock between the precast elements. This means that it is not necessary to take into account the orthotropic behaviour of the hollow core slab.

The main dimensions of the plate elements are 1.25 m in the longitudinally direction of the building and in the transverse direction 1.2 m, which is identical to the standard precast elements normally used in Denmark. The dynamic mass of the structure is assumed to be dead load and one third of the live load.

The modal analysis of the FEM model gives the results shown in table 1 and partly in fig. 4 for the eigenfrequencies.

Mode	Eigenfrequencies [Hz]	Mode	Eigenfrequencies [Hz]
1	4.80	6	7.33
2	5.47	7	7.79
3	5.74	8	8.70
4	6.24	9	9.63
5	6.96	10	10.04

Table 1. Eigenfrequencies of the floor structure.

It is obvious that mode 1 is very close to the simply supported one-way slab and therefore the first eigenfrequency 4.80 Hz is comparable to the previously calculated 5.17 Hz. The small difference is satisfactory and could mainly be explained by the fact, that the FEM model takes into account the small deformations of the cross beam and the vibration of the edge beam, which acting alone would have had a lowest eigenfrequency of 3.2 Hz. We can then conclude, that the FEM model is sufficiently exact.

Knowing the first 10 eigenfrequencies and the corresponding modes we are able from modal analysis to calculate the response of the floor structure being exposed to a forced dynamical load. For the structure in our example we have chosen to determine the response from people walking, running and jumping (dancing, gymnastic etc) as shown in table 2.

For the two load cases with one person moving it is assumed, that the movement takes place longitudinally crossing the mid span of the floor between two rows of columns near one of the facades. Jumping people are also assumed to act at the mid span. The area between the two interior rows of columns is in this building occupied by installations. The structure is conservatively assumed to have a critical damping ratio 0.01 for all modes.

Some of the results of the response calculations are shown in fig. 5 and 6.

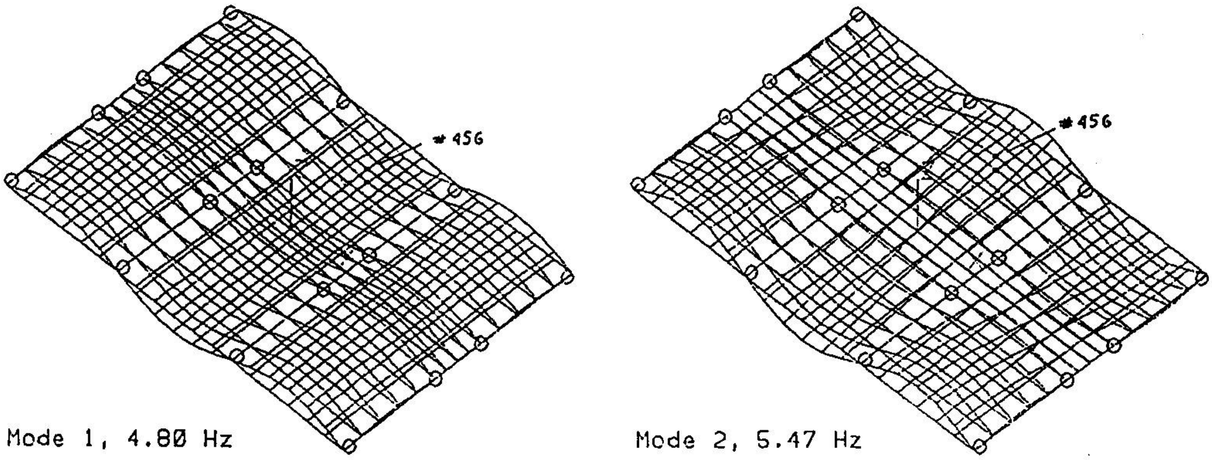

Figure 4. Vibration modes for the first 2 eigenfrequencies.

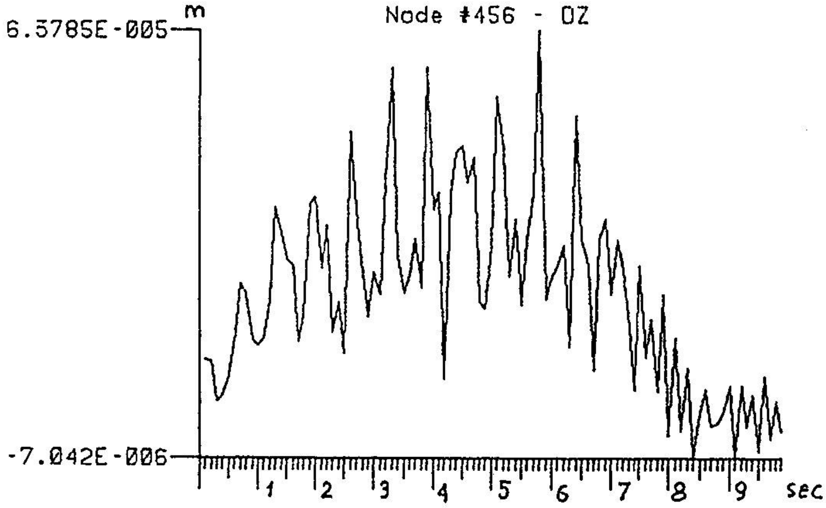

Figure 5. Vertical deformation (in m) of the mid span node (#456) versus time (in sec) during passing of a walking person.

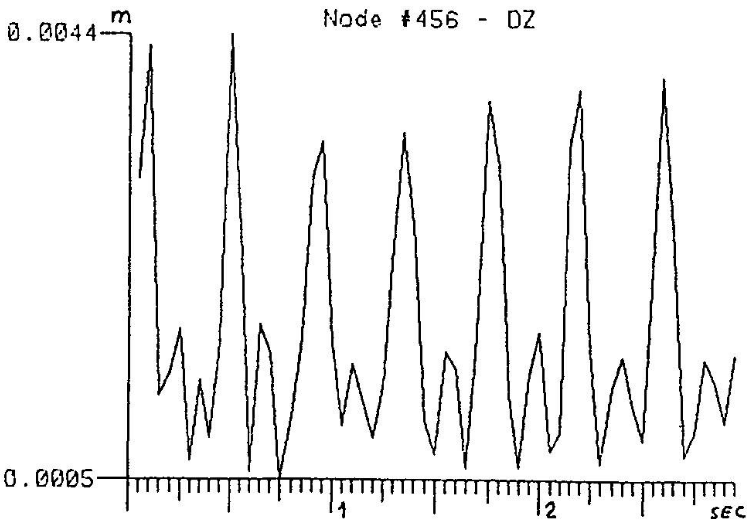

Figure 6. Vertical deformation (in m) of the mid span node (#456) versus time (in sec) during a dancing session (base frequence 2.4 Hz).

Dynamic load	Frequency [Hz]	Velocity [m/sec]	Static load
One person walking	1.6	1.0	800 N
One person running	2.4	2.9	800 N
People jumping (dancing)	1.6 2.4	0 0	1600 N/m² 1600 N/m²

Table 2. Dynamic loadings from human activities on the floor structure.

5. EVALUATION OF THE RESULTS

The response from one person walking across one slab is shown in fig. 5. The vertical deflection of the mid span node is max. 0.064 mm and the corresponding acceleration is max. 0.02 m/sec^2. The floor behaves very well to the walking person. For a running person we obtain a max. acceleration of 0.03 m/sec^2. When more than one person are walking the above acceleration will increase. An estimate of the increase factor is 6 [3], which means that the max. acceleration reaches 0.12 m/sec^2. All the results are below the acceptance level 0.2 m/sec^2 [3].

The response from people dancing or jumping with a frequency of 2.4 Hz is shown in fig. 6, where the deflection of the mid span node is shown. The max. acceleration of the slab at the dancing persons is 0.8 m/sec^2 which is acceptable (< 1 m/sec^2). The slab situated in the next span is vibrating with a dominant frequency of 4.8 Hz and the max. acceleration is 1.0 m/sec^2, which is considered acceptable for people participating in the event.

6. CONCLUSION

We conclude, that the floor structure behaves very well to normal and acceptable to abnormal induced vibrations from human activities. It can also be concluded, that the dynamic behaviour of a floor structure cannot be predicted alone by regarding it as composed by independent plate and beam units.

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Tuned Mass Dampers for Continuous Beams

Amortisseurs de masse pour poutres continues

Abgestimmte Tilger für Durchlaufträger

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SUMMARY

Structural serviceability of buildings also includes comfort of occupants during structural response to dynamic excitation. Excessive dynamic response compromising comfort and serviceability can be suppressed by means of tuned mass dampers. Attention is focused upon the case, where this technique is used to suppress dynamic vibration of continuous beams often characterized by closely spaced frequencies and modes. A theory is presented in which modal analysis plays the key role. Analytical expressions for the response are presented and applied to an example, thereby highlighting some important observations to be accounted for in the tuning strategy.

RESUME

L'aptitude au service de bâtiments comprend aussi le confort humain dans le bâtiment sous excitation dynamique. L'atténuation des oscillations excessives se fait souvent avec des amortisseurs de masse. Le cas d'une poutre continue, pour laquelle les fréquences souvent sont très serrées, est résolu avec plusieurs amortisseurs de masse. Les auteurs présentent une théorie opérationelle basée sur une analyse modale. Cette analyse détermine la réponse structurelle et un exemple est présenté pour souligner certaines observations importantes dans le cas d'amortissement optimal.

ZUSAMMENFASSUNG

Die Gebrauchstauglichkeit von Gebäuden umfasst viele Komponenten, von denen sich eine auf den Komfort der Benutzer während der Reaktion des Gebäudes auf eine dynamische Anregung bezieht. Eine wohlbekannte Methode zur Unterdrückung übermässiger dynamischer Reaktionen besteht in der Anwendung von abgestimmten Tilgern. Hier wird insbesondere auf den Fall von Durchlaufträgern eingegangen, die oft durch eng beieinander liegende Frequenzen und Modalformen gekennzeichnet sind. Es wird eine Theorie dargestellt, bei der die Analyse der Schwingungsmoden eine Schlüsselrolle spielt. Es werden analytische Formeln für die Reaktion vorgestellt; anhand eines Beispiels werden einige wichtige Erfahrungen hervorgehoben, die bei der Strategie für die Abstimmung berücksichtigt werden sollten.

1. INTRODUCTION

Dynamic structural response is an important issue related to serviceability of buildings. The source of dynamic excitation is environmental loads such as wind load on tall buildings or occupant activities, installed equipment and machinery. Development of stronger materials enhance dynamic serviceability problems and remedial actions aiming at not exceeding limit values for human exposure to vibrations [1], [2] often is called for. Many of the efficient remedial actions belong to the categories of active or passive damping. Tuned mass dampers belonging to the second category have been used with success [2], [3].

The concept of tuning [2] is well known for ODOF systems. This paper presents a theoretical and operational basis for optimum tuning strategies for continuous beams for which tuning involves more modes. Modal analysis is applied. Operating in the complex plane, explicit determination of the modal response of the damped system as well as the response of the dampers are presented. Through examples application of the theory is elucidated.

2. MODAL ANALYSIS OF CONTINUOUS BEAMS

The well known modal theory for continuous beams is briefly outlined in the following.
Using well known symbols the <u>free</u> vibration problem for beams is governed by

$$EI\frac{\partial^4 W}{\partial x^4} + m\frac{\partial^2 W}{\partial t^2} = 0 \tag{1}$$

Modes F(x) and corresponding cyclic eigenfrequencies ω are determined by (2) in which non-dimensional quantities have been introduced

$$A[F] - \lambda B[F] = 0 \quad , A[\ ] = \frac{d^4[\ ]}{d\xi^4} \ , \ B[\ ] = [\ ] \ , \ \xi = \frac{x}{L} \ , \ \lambda = \frac{m\omega^2 L^4}{EI} \tag{2}$$

Operators A and B are selfadjoint and defining $<f,g>$ as the integration from 0 to 1 of $f(\xi)g(\xi)$

$$<A[F],G> = <A[G],F> \ , \ <B[F],G> = <F,G> = <G,F> = <B[G],F> \tag{3}$$

Therefore modes $F_i(\xi)$, $F_j(\xi)$ corresponding to distinct eigenvalues λ_i and λ_j are orthogonal i.e

$$<A[F_i],F_j> = <B[F_i],F_j> = \int_0^1 F_i(\xi)F_j(\xi)d\xi = 0 \quad for\ i \neq j \tag{4}$$

Furthermore we are free to normalise modes with respect to the operator B, therefore

$$<B[F_i],F_j> = <F_i,F_j> = \delta_{ij} \quad , <A[F_i],F_j]> = \lambda_j \delta_{ij} \tag{5}$$

To illustrate the phenomena of closely spaced eigenfrequencies of continuous beams two examples are shown on fig. 1 and 2. A simple and efficient technique has been used to identify the modes - based on Fourier expansion of $F(\xi)$ with special modelling at the supports, [4].

Adding a load term to (1), we obtain the equation for <u>forced</u> vibrations

$$EI\frac{\partial^4 W}{\partial x^4} + m\frac{\partial^2 W}{\partial t^2} = p(x,t) \tag{6}$$

Modal expansion of W in the orthonormalized system of modes $F_j(\xi)$

$$W(\xi,t) = \sum_{j=1} a_j(t)F_j(\xi) \tag{7}$$

combined with (5), (6) determines $a_j(t)$. Assuming harmonic excitation $p_j(\xi,t)=p(\xi)e^{i\omega t}$ and including modal damping ζ_j, the modal components a_j are determined by

$$\frac{d^2a_j}{dt^2} + 2\zeta_j\omega_j\frac{da_j}{dt} + \omega_j^2 a_j = p_j^0 e^{i\omega t}\ ,\ p_j^0 = \frac{1}{m}\int_0^1 p(\xi)F_j(\xi)d\xi \tag{8}$$

Introducing the well known frequency response function and phase angle of the j's mode $\Phi(\Omega_{(j)},\zeta_j)$ and $\varphi_j(\zeta_j, \Omega_{(j)})$ where $\Omega_{(j)}=\omega/\omega_j$ the response can be expressed :

$$W(\xi,t) = W(\xi)e^{i\omega t}\quad ,\ W(\xi) = \sum_{j=1}\frac{1}{\omega_j^2}p_j^0\Phi(\Omega_{(j)})e^{i\varphi_j}F_j(\xi) \tag{9}$$

For ω sufficiently close to ω_i the response might become unacceptably high. Applying modal theory as used above, we extent the theory to include the effect of tuned mass dampers.

3. CONTINUOUS BEAMS WITH TUNED MASS DAMPERS

N_D mass dampers are placed at ξ_j, j=1, 2, ..N_D. The mass, damping and tuning frequency respectively is M_{Dj}, ξ_{Dj} and ω_{Dj}. The exiting frequency is ω and the response is determined by:

$$\frac{\partial^2 W}{\partial t^2} + \frac{EI}{L^4 m}\frac{\partial^4 W}{\partial \xi^4} = \frac{1}{m}p(\xi)e^{i\omega t} - \sum_{k=1}^{k=N_D}\mu_k\frac{d^2y_k}{dt^2}\delta(\xi-\xi_k) \tag{10}$$

$$\frac{d^2y_j}{dt^2} + 2\zeta_{Dj}\omega_{Dj}\Big(\frac{dy_j}{dt} - \frac{\partial W}{\partial t}\Big|_{\xi=\xi_j}\Big) + \omega_{Dj}^2(y_j - W_{\xi=\xi_j}) = 0 \tag{11}$$

δ is the Kronecker delta, $\mu_j=M_{Dj}/mL$ the mass ratio and y_j the movement of the j'th damper. Introducing (7), including N_m terms and using (5), - (10) and (11) take the form :

$$\frac{d^2a_j}{dt^2} + 2\zeta_j\omega_j\frac{da_j}{dt} + \omega_j^2 a_j = p_j^0 e^{i\omega t} - \sum_{k=1}^{k=N_D}\mu_k\frac{d^2y_k}{dt^2}F_j(\xi_k)\quad j=1,2,..,N_m \tag{12}$$

$$\frac{d^2y_j}{dt^2} + 2\zeta_{Dj}\omega_{Dj}\Big(\frac{dy_j}{dt} - \sum_{k=1}^{k=N_m}\frac{da_k}{dt}F_k(\xi_j)\Big) + \omega_{Dj}^2\Big(y_j - \sum_{k=1}^{k=N_m}a_kF_k(\xi_j)\Big) = 0\quad ,\ j=1,2,...,N_D \tag{13}$$

We write the solution in the form (14) and make the variable transformation (15), (16):

$$a_j = a_j^0 e^{i(\omega t+\varphi_j)} \quad , \; y_j = y_j^0 e^{i(\omega t+\psi_j)} \tag{14}$$

$$j=1 \,..\, N_m : \quad z_{2j-1} = a_j^0 \cos\varphi_j \quad , \; z_{2j} = a_j^0 \sin\varphi_j \tag{15}$$

$$j=1 \,..\, N_D : \quad z_{2N_m+2j-1} = y_j^0 \cos\psi_j \quad , \; z_{2N_m+2j} = y_j^0 \sin\psi_j \tag{16}$$

(14) to (16) is introduced into (12) and (13) together with the definitions

$$F_{jk} = F_j(\xi_k) \quad , \; \Omega_{(Dj)} = \frac{\omega}{\omega_{(Dj)}} \qquad : j=1 \,..N_m \;, \; k=1 \,..N_D \tag{17}$$

whereby $N=2(N_m+N_D)$ equations for the N unknown z_i have been determined :

<u>$1 \le j \le N_m$</u>

$$(1-\Omega_{(j)}^2) z_{2j-1} - 2\zeta_j \Omega_{(j)} z_{2j} - \Omega_{(j)}^2 \sum_{k=1}^{k=N_D} \mu_k F_{jk} z_{2N_m+2k-1} = \frac{p_j^0}{\omega_j^2} \quad ...eq(2j-1)$$

$$2\zeta_j \Omega_{(j)} z_{2j-1} + (1-\Omega_{(j)}^2) z_{2j} - \Omega_{(j)}^2 \sum_{k=1}^{k=N_D} \mu_k F_{jk} z_{2N_m+2k} = 0 \quad ...eq(2j)$$

<u>$1 \le j \le N_D$</u>

$$-\sum_{k=1}^{k=N_m} F_{jk} z_{2k-1} + 2\zeta_{Dj} \Omega_{(Dj)} \sum_{k=1}^{k=N_m} F_{jk} z_{2k} + (1-\Omega_{(Dj)}^2) z_{2N_m+2j-1} - 2\zeta_{Dj} \Omega_{Dj)} z_{2N_m+2j} = 0 \quad ...eq(2N_m+2j-1)$$

$$-2\zeta_{Dj} \Omega_{(Dj)} \sum_{k=1}^{k=N_m} F_{kj} z_{2k-1} - \sum_{k=1}^{k=N_m} F_{kj} z_{2k} + 2\zeta_{Dj} \Omega_{(Dj)} z_{2N_m+2j-1} + (1-\Omega_{(Dj)}^2) z_{2N_m+2j} = 0 \quad ...eq(2N_m+2j)$$

$$\textit{That is}: \qquad K_{ij} z_j = b_i \tag{18}$$

after having solved these for z_i (note K_{ij} is <u>not</u> symmetric) we find for instance:

$$a_j^0 = sign(z_{2j-1}) \sqrt{z_{2j-1}^2 + z_{2j}^2} \quad , \; \varphi_j = Arctan\left(\frac{z_{2j}}{z_{2j-1}}\right) \quad : j=1 \,..\, N_m \tag{19}$$

and for the response (similar expression for the damper displacement) :

$$W(\xi,t)=W(\xi)e^{i\omega t} \quad , \quad W(\xi)=\sum_{j=1}^{j=N_m} a_j^0 e^{i\varphi_j} F_j(\xi) \tag{20}$$

3.1 Application, example

The beam shown in fig. 1 and 3 must be damped for exiting frequencies ω in an interval including the four lowest modes. The analysis includes 12 modes i.e. $N_m=12$. The number of dampers are 4, hence $N_D=4$ and N in (18) is $=2(12+4)=32$. Having determined the 12 modes according to the method briefly outlined in section 1, p_j^0 and the coefficients F_{jk} are determined. ω and the chosen tuning frequencies ω_{Di} determines $\Omega_{(j)}$ and $\Omega_{(Dj)}$. At this point - for each exiting frequency - K_{ij} and correspondingly all information regarding the response can be extracted from (18) to (20).
The tuning strategy must according to our experience observe the following rules: 1) Mass damper j, aiming at suppressing resonant response of mode j, should preferably be placed where mode no. j has it maximum value. 2) In addition to obeying rule no 1, care should be taken not to place dampers damping modes ω_i and ω_{i+1} close to each other as the dampers then will be working in opposite phase and cancel each other in the frequency range ω_i to ω_{i+1}. For instance if damper D2 is moved close to damper D1 and D4 is moved to a position near D2 we still obey rule no 1, but the damping arrangement would be very inefficient in the frequency range $\omega=$ 1.00 to 1.15.

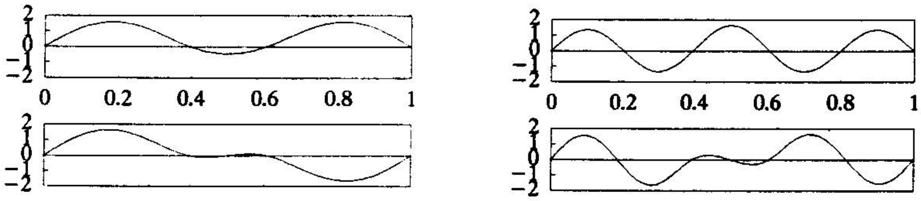

FIG. 1 Continuous beam over two internal supports. Length of span 1 and 3: 0.39474 L and of span 2 0.21052 L, where L is the total length. Shown from top to bottom - from left to right orthonormalized modes no. 1 to 4, which are lumped in pairs: $\sqrt{\lambda_i}$ = 76.8, 87.1 and 246, 288 for i=1 to 4. $\lambda_i = m\omega_i^2L^4/EI$.

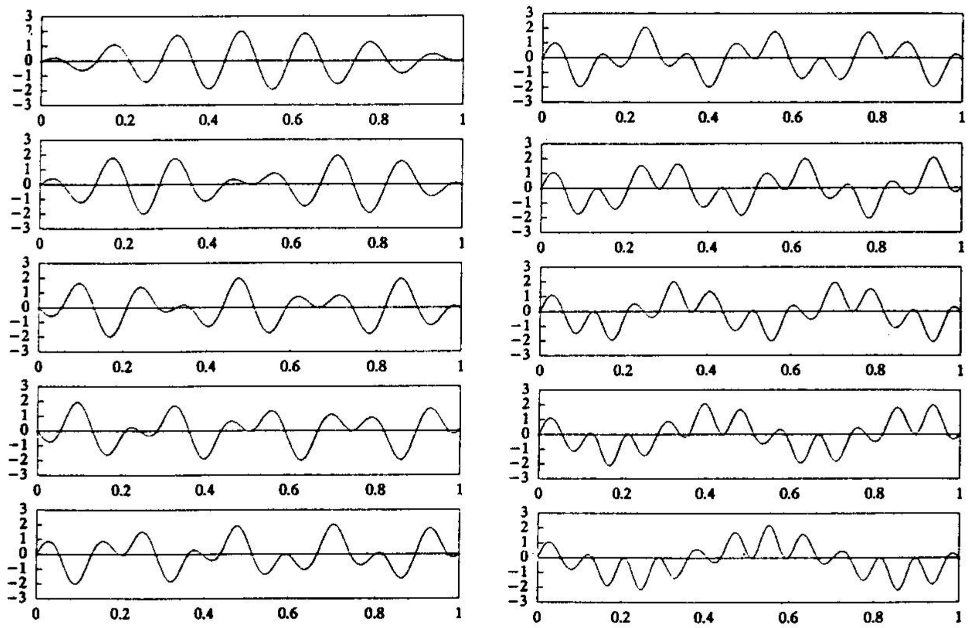

FIG. 2 Continuous beam over 13 internal supports. Length of span 1: 0.05536 L, length of span 2 to 13: 0.07631 L length of span 14: 0.028865 L. Shown from top to bottom from left to right orthonormalized modes 1 to 10. Eigenfrequencies only differ by a factor 2 from mode 1 to mode 10: $\sqrt{\lambda_i}$ = 1723, 1805, 1934, 2103, 2302, 2524, 2762, 3010, 3257 and 3492 for i=1 to 10. $\lambda_i=m\omega_i^2L^4/EI$

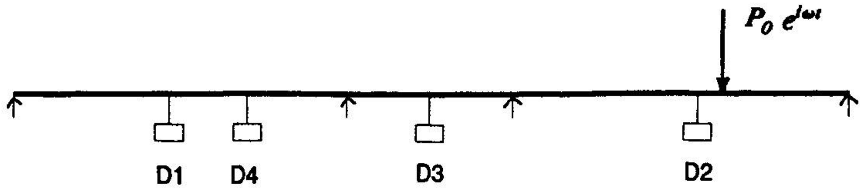

FIG. 3 Beam of fig.1. Single force acts at ξ =0.85 at exiting frequencies $0 \le \omega \le 4$. Four tuned mass dampers placed at ξ_1=0.185, ξ_2=0.82, ξ_3=0.50, ξ_4=0.28. 12 modes have been included in the analysis: ω_i, i=1 to 12: 1.000, 1.135, 3.202, 3.750, 5.016, 7.858, 8.397, 12.75, 14.08, 15.99, 21.28 and 22.16.Modal damping ζ_i=0.01. The following tuning frequencies, mass ratio and damping values for the tuned mass dampers have been applied: ω_{Dj}=0.99, 1.124, 3.17 and 3.713. μ_{Dj}=2% and for ζ_{Dj} (j=1 to 4) two set of values have been used: 0.15 and 0.30.

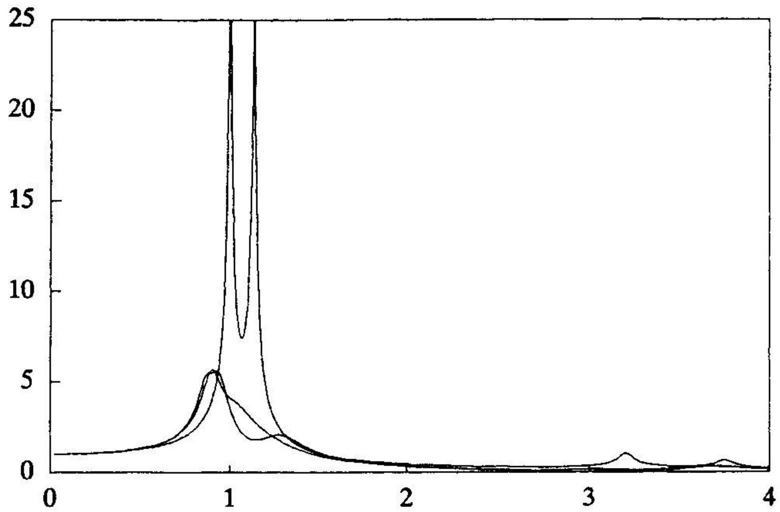

FIG. 4 Frequency response for the continuous beam of fig 1 and 3. x- axis frequency - y axis: the amplitude of the beam at the point where the load is applied. The two lower curves show the response for the damper configuration of fig 3 - data as given in legend to fig 2. The dampers reduce the peak values of the undamped system by a factor 5 to 10.

4. CONCLUDING REMARKS

A theory applicable to the case of dynamical exited beams being damped with tuned mass dampers has been presented. Based on the theory a user friendly edp based numerical tool enabling the designer to implement the optimum damping arrangement will be developed. The mass ratio to be applied to obtain efficient damping is very small (2% or less) - therefore the mechanical devices acting as tuned mass dampers can be very simple. Another potential avenue for future work of course will be to propose a simple design for such devices.

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Floor Response from View of Human Body and Construction

Comportement des planchers du point de vue physiologique et constructif

Deckenverhalten in physiologischer und konstruktiver Hinsicht

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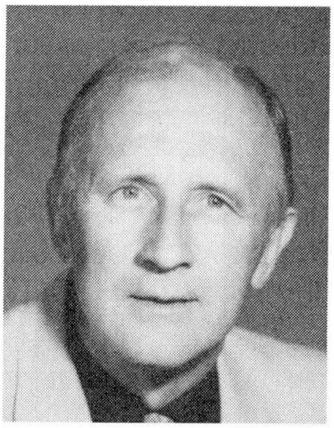


Miroš Pirner, born in 1928, civil engineer from the Techn. Univ. Prague 1952. Worked till 1970 as assistant professor, five years as research worker. In 1976–1990 head of the department of statics and dynamics at the Res. and Testing Inst. of Buildings. Since 1990 director of the Institute of Theoretical and Applied Mechanics.

Ondřej Fischer, born in 1929, got his civil engineering degree at the Techn. Univ. of Prague. Worked as a teaching assistant for 10 years, 3 years at practical design of civil engineering structures, since 1964 in the Czechoslovak Academy of Sciences. Concerned with scientific research in the field of dynamics of structures.

SUMMARY
The serviceability of structures with reference to their effect on the human organism is determined by the respective Codes. The authors have collected the results obtained by 15 years of measurements of response of the buildings of different systems, different materials and subjected to different dynamic loads. The authors present one method for the limitation of the vibrations of a textile factory building by means of a system of additional masses and rubber insulators.

RESUME
Des normes traitent de l'aptitude au service de constructions en fonction de l'effet que peuvent avoir les vibrations sur la sensibilité de l'organisme humain. Les auteurs exposent les résultats qu'ils ont recueillis au cours de 15 années de mesures sur les vibrations des bâtiments constitués de différents systèmes et matériaux et soumis à différentes actions dynamiques. Ils proposent une méthode de limitation des vibrations dans un ancien bâtiment servant d'atelier textile, en lui substituant un système de masses additionnelles posées sur des appuis élastiques en caoutchouc.

ZUSAMMENFASSUNG
Die Ausnutzbarkeit der Konstruktionen in Bezug auf die Empfindlichkeit des menschlichen Organismus für Schwingungen wird durch Normen vorgeschrieben. Die Autoren präsentieren ihre in 15 Jahren gesammelten Ergebnisse von Messungen der Schwingungen von Gebäuden verschiedener Systeme, Baustoffe und dynamischer Lasten. Es wird vorgeschlagen, die Schwingungen eines alten Fabrikgebäudes durch ein System von Zusatzmassen auf elastischen Gummilagern zu vermindern.

1 Introduction

The level of vibrations transferred to human body has proved to be a very important component of environment condition, influencing significantly the man's health, the quality of his production and the sense of comfort. Also the Czecholovak hygienic rules [2], [3] give admissible values of accelerations, differenciating living and working places, the type of the work and the time of exposure to the vibrations. In the same way as the importance of the environment requirements has grown during last years, it has also increased the number of contentions and the vibrations had often to be objectively appraised; we have therefore measured several tens of buildings and some of their results are summarized in Fig. 3.

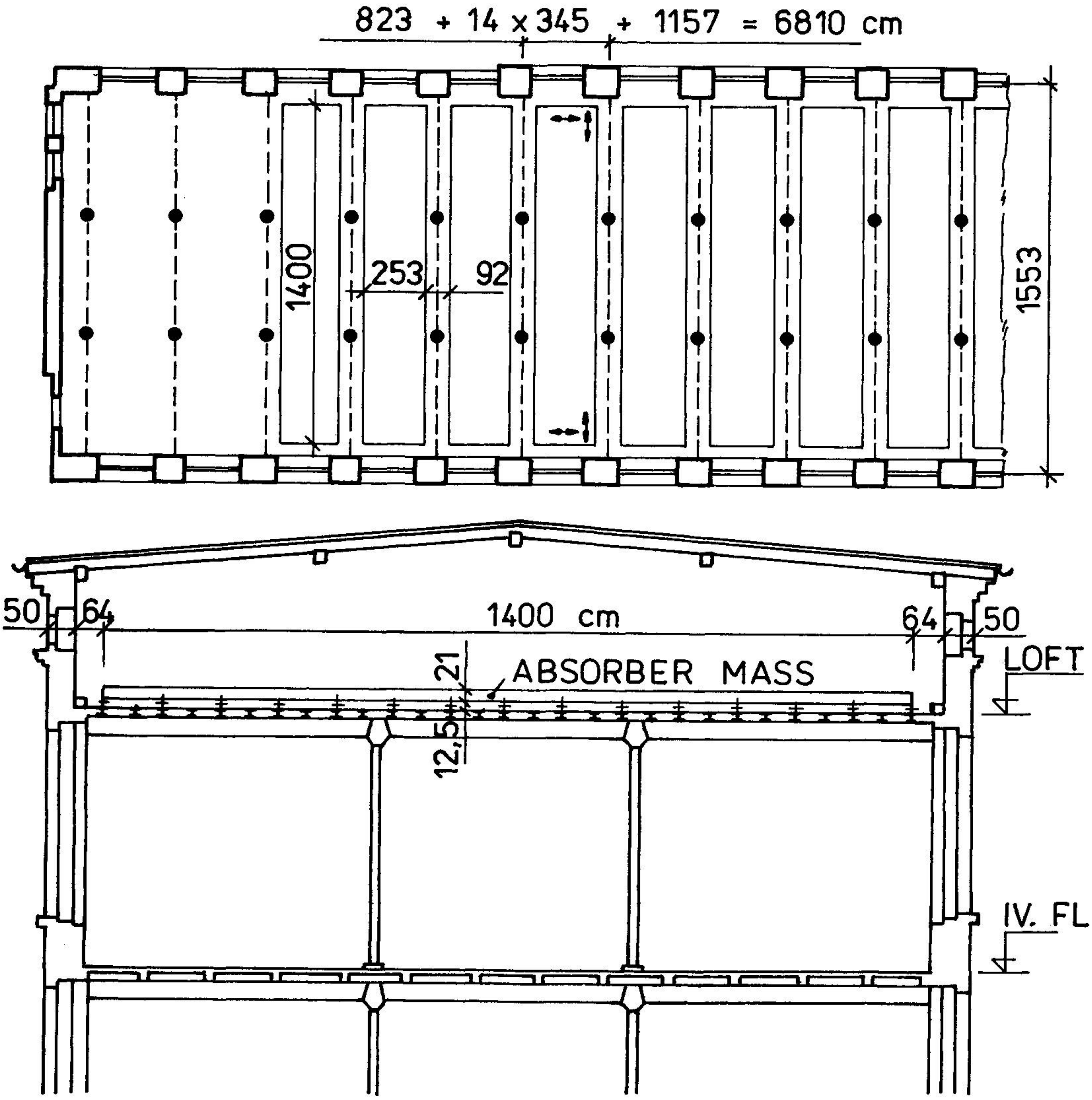


Fig. 1 Cross section and ground plan of the building

The methods of limitation of horizontal movements of tall buildings with very low natural frequencies (less than 1 cps) using big masses moved by hydraulic jacks are well known [1]. Here we present a more simple example of limitation of horizontal vibrations of one old textile factory building by means of the passive tuned mass damper, i.e. by a system of additional masses supported by rubber elements.

2 The behaviour of the building

The factory building under consideration is cca 100 years old, its groundplan and cross section are given on the Fig. 1. The building has 4 floors of reinforced concrete slabs (made later, in substitution of the original wooden ceilings), supported by two rows of cast iron columns. The outer walls are from bricks. On each floor there were 80 looms the operation of which excited the building in Y (longitudinal) direction, the frequency was 2.60 cps. The measured horizontal amplitude of the 4th (highest) floor was 0.3 mm, the natural frequency of the fundamental mode (Fig. 2) was 2.50 cps, the logarithmic decrement $\delta = 0.09$. The measured values of the displacement, velocity and acceleration are signed as point "12" in Fig. 3: the two boundaries depict the admissible limit values for brainworkers and for other workers after [2].

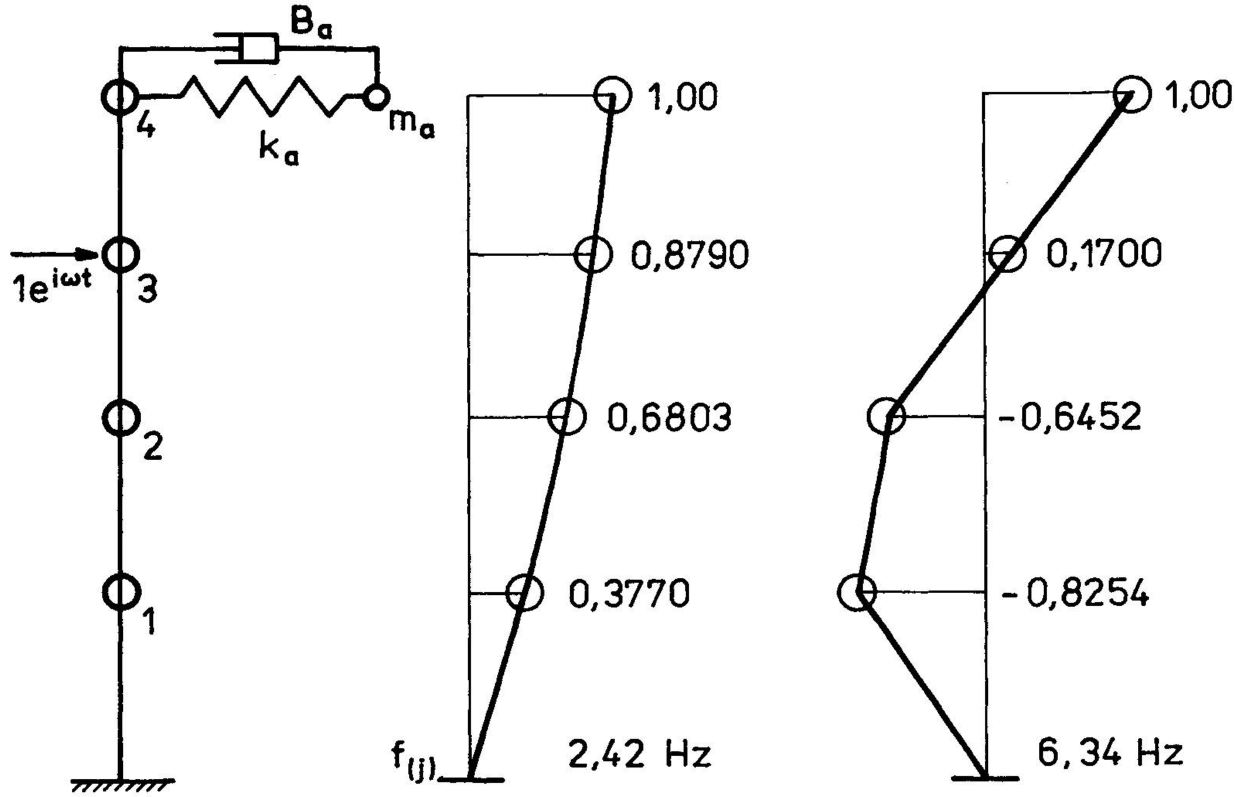


Fig. 2 Mechanical scheme of the structure and the calculated natural modes

The biggest measured value of the velocity of the response does not differ much from 5 mm/s, which is the limit (after [4]) for the appearence of the first damages of the structure. In spite of these large amplitudes and of the long duration of the vibrations the building was not cracked. Newertheless the walking on the 4th floor was difficult and that is why the possibilities of reducing the vibrations were seeked.

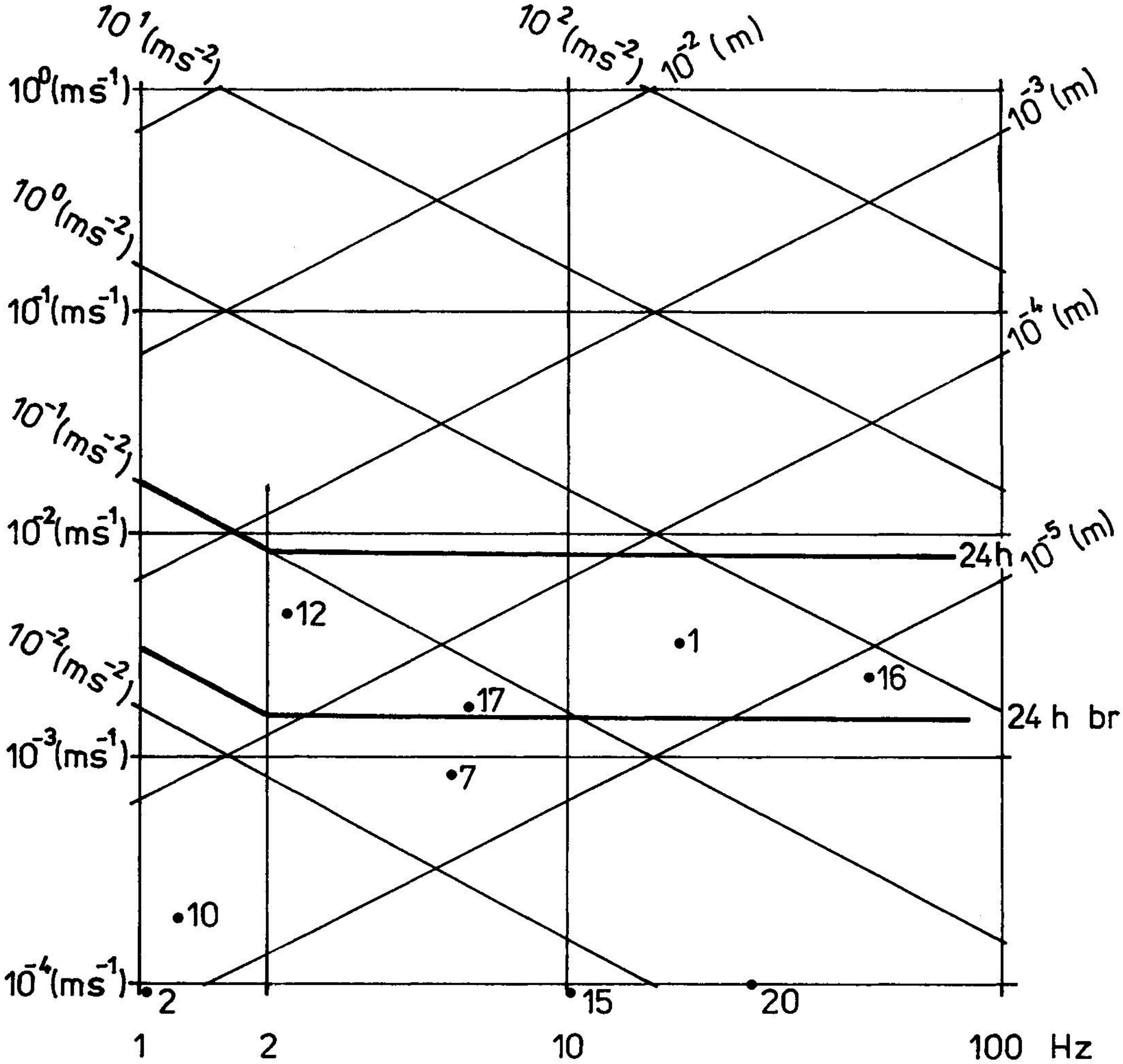
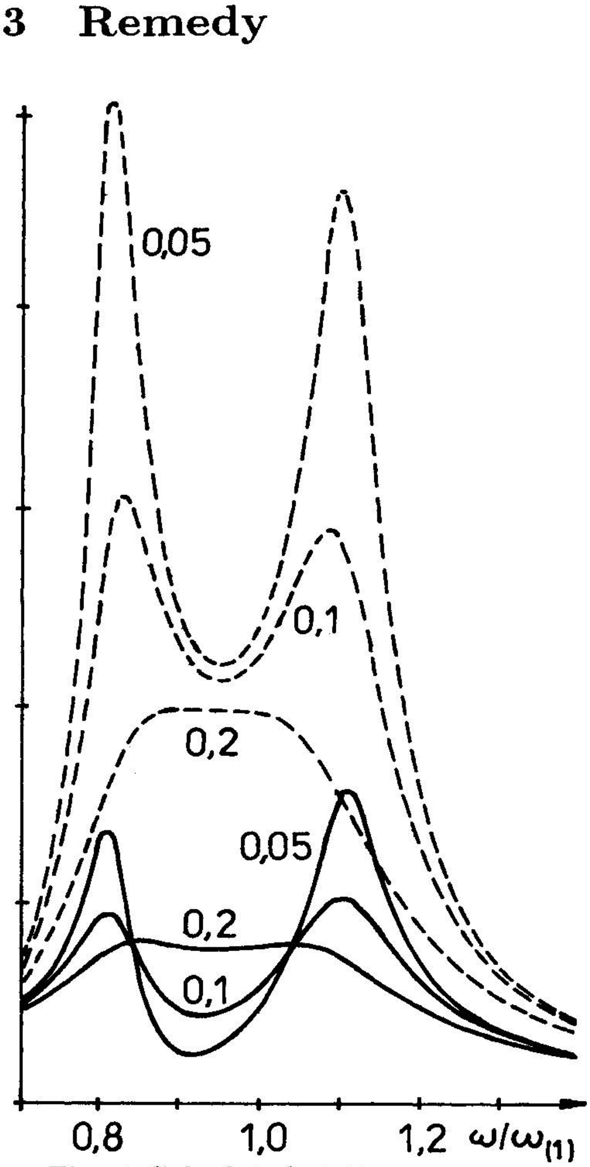


Fig. 3 Graphic representation of horizontal vibrations measured on some buildings. The solid line "24 h" shows the admissible limit for 24 hours of exposure, the line "24 h br" the same limit for brainworking personnel [2].

1 - Brick building, excited by tram
2 - Brick building, excited by road traffic
7 - Brick family house, excited by the operation of a saw mill (5 cps)
10 - Mining tower in normal operation
12 - Factory building described in this paper
15 - Concrete building, excited by street traffic
16 - Concrete building, excited by the operation of a piston compressor
17 - steel building, excited by the normal movement in the building

3 Remedy



After experiences with successful applications of pendulum absorbers on television towers and guyed masts [5] the authors designed a dynamic vibration absorber also for this case. The absorber should be installed on the loft of the building. It should consist of 14 horizontal slabs - steel basins 14.0 x 2.53 x 0.21 m filled with sand and covered by concrete shell. Each of these masses (14.30 t) was placed on 30 rubber bearings - cylinders of 100 mm dia and 65 mm heigt; the masses coud move horizontally while the rubber worked in shear. The system was modelled for the calculation as a shear-type building with 4 lumped masses; the 5th mass, that of the absorber, was attached to the 4th mass representing the ceiling of the highest floor and the roof (see Fig. 2). The attachment of the absorber has been modelled by a spring and damper according to the properties of the rubber elements, given by the producer. The system was calculated for harmonic excitation $F(t) = F_o \exp(i\omega t)$ acting on the mass m_3 (the floor of the workshop with the looms). From the shape of the resonance-curves (stationary amplitude of the response of the mass m_3 plotted against the frequency of the excitation ω - example see Fig. 4) the optimum type of the rubber bearing could be chosen. The construction of rubber supports of the absorber masses is shown on Fig. 5

Fig. 4 Calculated stationary response of the building with the absorber (solid line) and relative amplitude of the absorber mass (dashed line) for different damping β in the supports.

4 Conclusion

The author regret that the design of the described dynamic absorber could not be realised in spite of its simplicity and theoretical clearness. The meticulous apprehension of the management of the factory to invest a modest amount of money to a non typical device has caused the suspension of the project and the building was let to continue the vibration.

The arrangement and the construction of the absorber was mostly designed by Mr Zdeněk Patrman, whose helpful cooperation is gratefully acknowledged.

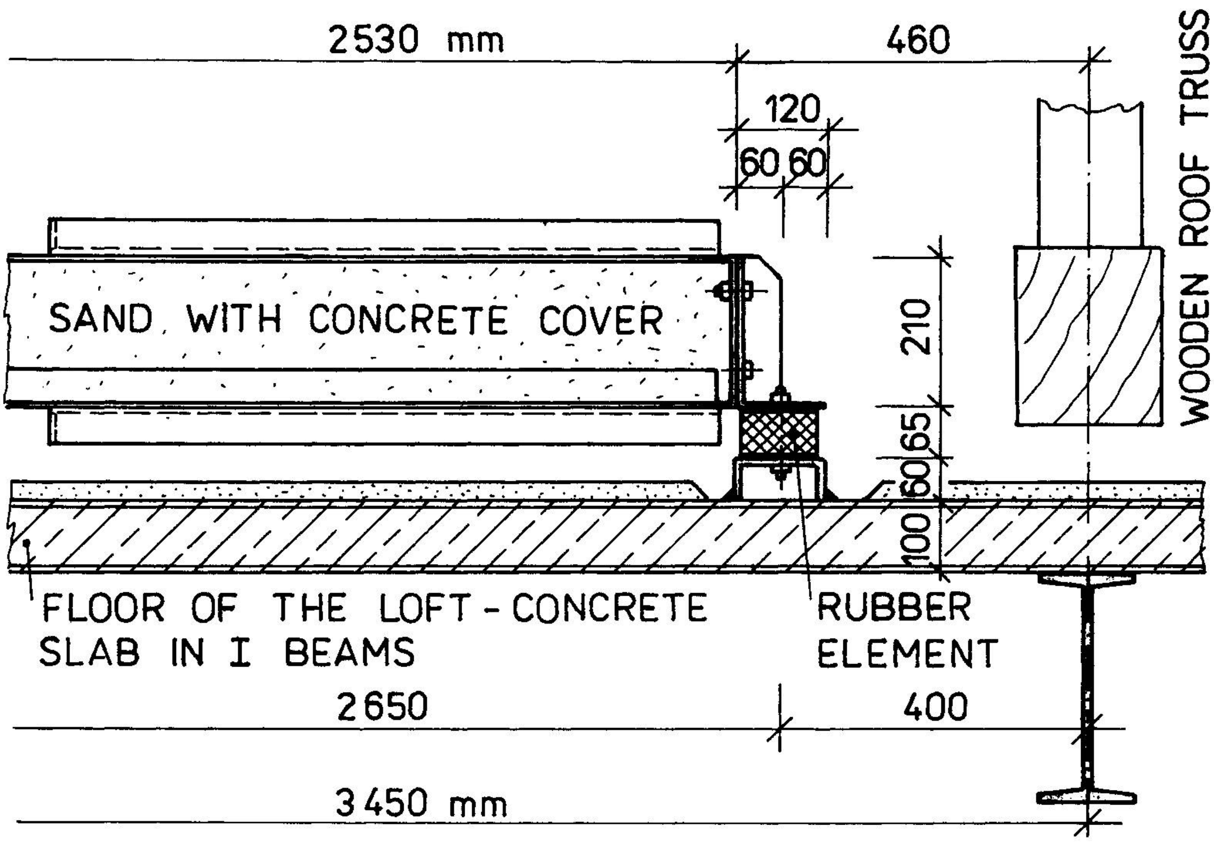


Fig. 5 Detail of one of the 30 rubber supports of one absorber mass

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Vibration and Serviceability in Post Terminal Buildings

Vibrations et aptitude au service dans les bâtiments postaux

Gebrauchsfähigkeit und Schwingungen in Postabfertigungsgebäuden

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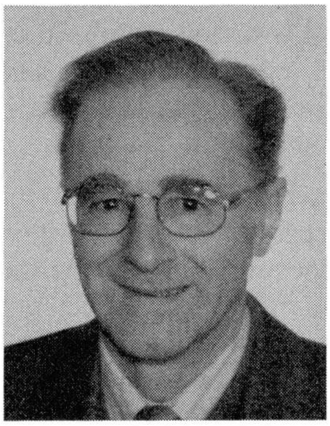
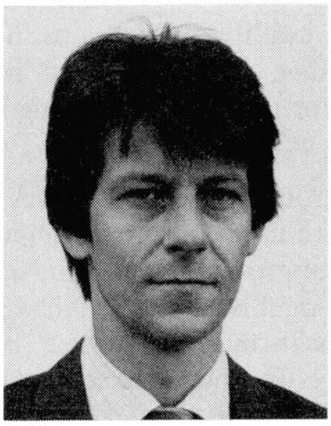


Harald Dahlsjö, born 1932, received his civil engineering degree at Chalmers Univ. of Technology 1957. Since then he has been active in structural design of many kinds of buildings. He is presently responsible for structural design at the Swedish Post Real Estate Management.



Sven Ohlsson, born 1952, received his civil eng. degree at Chalmers Univ. 1977, where he also received his doctoral degree and later was acknowledged the title associate professor. He has been active in structural research focused on dynamic problems and is presently in charge of the unit for Dynamics in Design at Chalmers.

SUMMARY

Buildings intended for mixed activities, such as Swedish post terminals, need special attention to assure a serviceable indoor environment. Vibration is a potential serviceability problem and it is adressed here. Typical activities in post terminals are reviewed and common structural configuration is described. Forklift transportation and handling of goods cause dynamic service loads. Past and presently undertaken experimental studies show that adequate structural systems, smooth floor surfaces and proper design are essential to avoid annoying vibrations. The need for better knowledge about dynamic design loads is urgent.

RESUME

Les bâtiments destinés à des activités variées, tels que les bâtiments postaux suédois, requièrent une attention particulière afin d'offrir de bonnes conditions de travail. Les vibrations représentent un problème potentiel pour l'aptitude au service. Les activités habituelles dans les bâtiments postaux et les dispositions constructives types sont présentées. Le transport par chariot élévateur et le traitement des marchandises provoquent des charges dynamiques. Etudes et expériences montrent que des systèmes structuraux adéquats, des surfaces de plancher molles et un projet bien conçu sont essentiels pour éviter les vibrations désagréables.

ZUSAMMENFASSUNG

Gebäude, die für gemischte Aktivitäten vorgesehen sind, wie schwedische Postabfertigungsanlagen, brauchen besondere Aufmerksamkeit, um gute Arbeitsverhältnisse zu garantieren. Schwingungen geben ein mögliches Problem der Gebrauchstauglichkeit, das hier behandelt wird. Typische Aktivitäten in Postterminals werden übersichtlich diskutiert und gewöhnliche Tragwerkslösungen werden beschrieben. Gabeltransporter und Güterabfertigung verursachen dynamische Gebrauchslasten. Experimentelle Studien zeigen, dass zweckmässige Tragwerkssysteme, weiche Fussbodenflächen und gebrauchsgerechte Konstruktionslösungen wesentlich sind, um störende Schwingungen zu vermeiden.

1. INTRODUCTION

Vibration as a potential source of reduced serviceability is described in relation to buildings which are supposed to accommodate multiple activities. The building type in focus - Swedish post terminal buildings - typically contains light industrial activities including indoor traffic as well as offices and even separate areas for physical fitness training. Typical activities and structural systems will be described. Short summaries of dynamic studies will also be given.

2. POST TERMINAL BUILDINGS

2.1 Activities

2.1.1 Post handling activities

A general overview is given in [1]. The activities at Årsta post terminal in Stockholm are described here as an example. 35 000 post items arrive daily. The post is mainly delivered by lorries to the terminal. It is contained in mailbags and wheeled post containers. The containers are used for mailbags containing letters and for parcels. The total terminal area is 37500 m^2 distributed between five stories. Two stories are used for post handling activities.

The ground floor includes a loading dock for lorries and two machine sorting lines for arriving mail. They are used for a first preliminary sorting with respect to letter size and item type. Furthermore the ground floor includes three machine sorting lines for mail leaving the terminal. The sorted mail is put into mailbags or post containers. One of these lines includes a robot station where sorted post parcels (boxes) are automatically put into containers. The first floor accomodates five machine sorting lines and one which is manually operated.

Post containers are continuously (day and night) transported across the ground floor to different stations by 8 forklift trucks. Each forklift will typically carry out 150 - 200 transportation tasks daily. A forklift weighs 27 kN and a post container adds a weight of 3 to 8 kN. The post is transported between the ground floor and the first floor by use of overhead conveyors. Except for forklift drivers, 15 persons are working on each floor at a time.

Fig. 1 Ground floor view

2.1.2 Office areas

There is a need for a foremans office where planning activities and computer support are based. These office rooms need to be located close to the post handling activities. The solution has been to accomodate this office on a mezzanine deck, which is located between the ground floor and the first floor and suspended from the first floor. This office space may experience vibration which may reduce the serviceability.

No complaints about vibrations have been noted from the more regular office areas in the terminal.

2.1.3 Physical training area

Several of the post terminals include an area which is planned for physical training. Such physical excercises typically occur during office hours. This severe dynamic loading may result in annoying vibration if this aspect is overlooked in the structural design, cf. references [2] and [3].

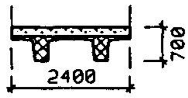
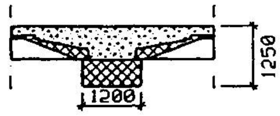
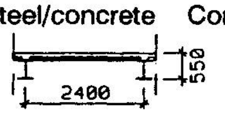
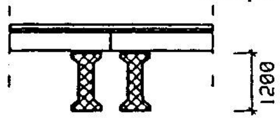
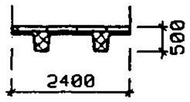
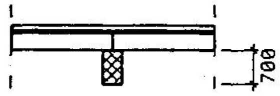
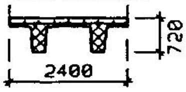
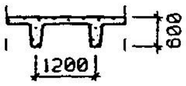
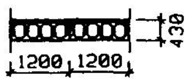
2.1.4 Activity - response cases

A typical terminal is supposed to accomodate a variety of activities, some of which are expected to induce substantial dynamic loads under service conditions. The most important dynamic load sources are vehicle loads from forklifts, goods handling loads from loading and reloading post containers and footstep forces from physical training. Humans are the most important critical sensors of resulting vibration. Other sensitive objects include automatic equipment for weighing and sorting letters.

2.2 Typical structural design of post terminals

The building has typically a beam-column type structure. Columns are often supporting floor bays of, say 12 x 12 m^2 unobstructed area. Various examples of cross sections for floors supporting forklift truck traffic and fitness training, respectively, are given in Table 1.

Table 1. Sample floor designs - Forklift goods handling and fitness training

Terminal	Deck structure Type	Deck structure Span (m)	Girder Type	Girder Span (m)	Mass (kg/m^2)	f_1 (Hz) Calc.	f_1 (Hz) Meas.	Employee reaction
Tomteboda ground floor (1982)	PC TTK240/50 + 200mm RC 	12.0 Cont.	PC RB120/50 + RC 	13.0 Cont.	1000 (Deck: 830)	7.5	-	None (forklift traffic floor)
Tomteboda sec. floor	Composite steel/concrete 	12.0 Cont.	PC I-beams 	16.8 Simpl.	600 (Deck: 410)	4.7	-	Some (forklift traffic floor)
Borås (1988)	PC TT240/40 + 100mm RC 	10.0 Simpl.	PC RB40/70 Rect beams 	8.0 Simpl.	580 (Deck: 510)	7.9	-	Some (forklift traffic floor)
Norrköping (1991)	PC TTK240/60 + 120 mm RC 	14.0 Cont.	PC FB70/70 + RC 	6.0 Simpl.	780 (Deck: 680)	8.7	10.0	None (forklift traffic floor)
Årsta (1991)	RC TTSwedeck 	12.0 Cont.	Steel girders concr covered 	10.7 Cont.	550 (Deck: 500)	6.8	7.8	Some (forklift traffic floor)
Linköping (1992)	PC HD120/32 + 140 mm RC 	10.0 Simpl.	PC FB70/80 	12.0 Simpl.	800 (Deck: 740)	6.5	-	New build. (physical training floor)
Västerås (1992)	PC HD120/38 + 50 mm RC 	12.0 Cont.	Steel HSQ55/35 	9.0 Simpl.	600 (Deck: 600)	5.0	-	None (physical training floor)

3. VIBRATION STUDIES OF THE ÅRSTA POST TERMINAL

3.1 Experimental program

The post terminal at Årsta was completed in 1991. Dynamic measurements were conducted at two different times. The first set of measurements was carried out during the summer of 1989. The main parts of the building structure were then erected. A second set of measurements was made in January 1991, when the building was finished. The studies are reported in [4]. The following three basic types of dynamic tests were carried out:

a. Forced vibration tests with subsequent limited experimental modal analysis aiming at modal parameters such as resonance frequencies f_n, modal damping ratios $(c/c_{cr})_n$ and mode shapes Φ_n for some of the lower floor vibration modes n.

b. Vibration tests with simulated service loads from forklift operations aiming at estimates of floor vibration levels related to different goods handling activities and forklift types.

c. Vibration tests using different dynamic loads aiming at establishing dimensionless transfer functions between different floor areas and between floors belonging to different stories.

Besides these three types of testing with corresponding aims, there were two benefits from repeating the testing at two different times. It enabled comparisons between a construction stage ('clean' structure) and a completed stage and between a relatively fresh stage and a later stage were some cracking and increase in Youngs modulus for concrete could have been expected.

3.2 Sample measurement records and results

A value for the fundamental frequency $f_1=7.8$ Hz was found for the floor area with 12 m span intended for forklift activity [4, 1989]. The second test series showed the same value for the fundamental frequency [4, 1991]. By 'the same' is meant identical within the two-digit accuracy. Higher modes were closely spaced, $f_2=8.5$ and $f_3=9.0$ Hz. The values for the experimentally measured natural frequencies are in fairly good agreement with theoretically calculated values based on the assumption about fully effective floor element cross sections (no cracks). Damping ratios c/c_{cr} for corresponding modes were evaluated to 1.3%, 1.2% and 1.7% respectively [4, 1991]. These values are rather typical (including the scatter) for this type of construction.

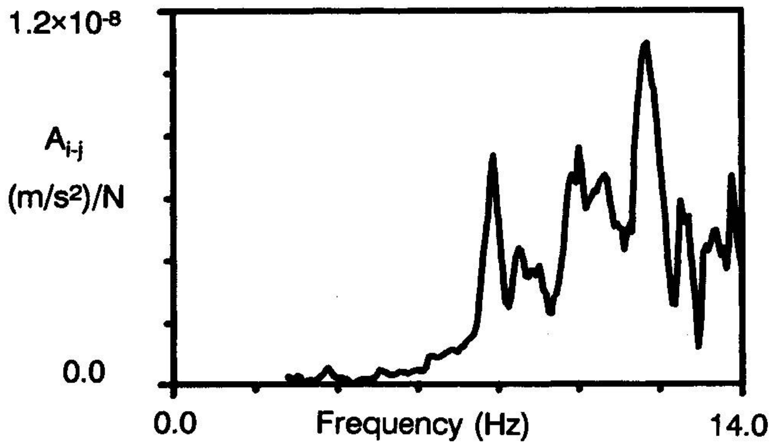


Fig. 2 Magnitude of accelerance function $A_{i\text{-}j}$ for two locations i,j at ground floor of the Årsta post terminal

Two conclusions may be that the floor area in question is basically uncracked even after it had experienced some static and dynamic service loads and that no substantial effective increase in concrete stiffness has occurred due to ageing. The risk of future cracking and corresponding reduction in resonance frequencies may of course not be neglected. The experimental values found for damping ratios support the previous suggestion of $c/c_{cr}=1\%$ for design purposes.

The vibration tests which utilized forklift traffic as a simulated service load were carried through for a number of prescribed driving paths and manoeuvres. They also included two different forklift types. Examples of acceleration measurements at the ground floor in the

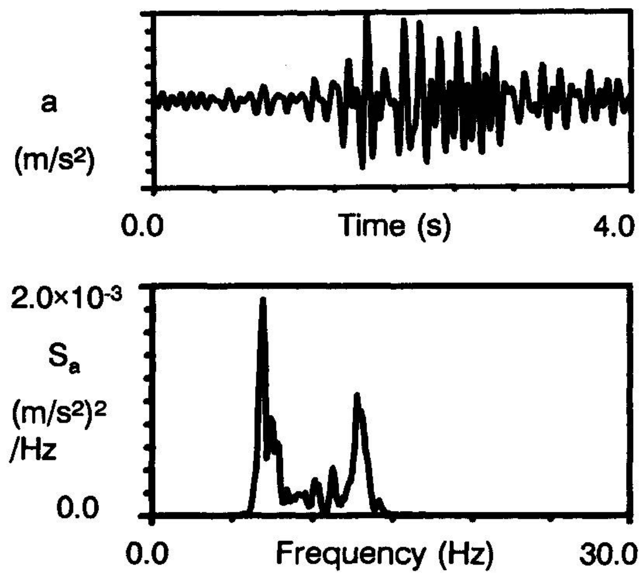


Fig. 3 Acceleration time record (±0.5 m/s²) and corresponding acceleration spectral density S_a for ground floor due to regular driving of a forklift of type AT.

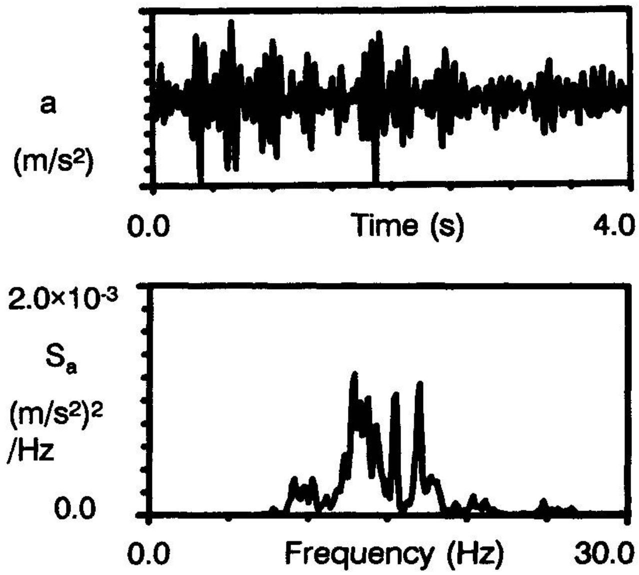


Fig. 4 Acceleration time record (±0.5 m/s²) and corresponding acceleration spectral density S_a for ground floor due to regular driving of a forklift of type Rocla.

vicinity of the driving area for the forklift are presented in Figs. 3 and 4 (after [4] 1991). It is worth noting that the softer wheels of the forklift of type AT result in a low-frequency dominated acceleration spectrum, basically limited to the frequency band 5 - 15 Hz, while the corresponding frequency range for the forklift type Rocla is aproximately 8 - 30 Hz. From Fig 3. it is clear that the spectral function shows a pronounced peak around 6.5 Hz. Closer studies have shown that this is not an effect of a floor resonance, but rather originates from the dynamic characteristics of the vehicle. This peak appears in vibration spectra for different floor areas and it changes only when the load carried by the forklift is altered.

Vibration levels of some sort must be established in order to estimate the possible violation of full serviceability. The ISO standard [5] is probably the document which is most widely acknowledged in this context. It recommends the use of frequency-weighted rms values for vibration acceleration as the main vibration parameter representing the annoying effect on people. Such values are, however, strongly depending on the averaging time used. The time dependence certainly needs more research attention. A time interval of 1 minute was chosen in [4] for such averaging. A total of maybe an hour of acceleration recordings was taken. Subsequent processing identified a minute for each loading case and floor area which yielded the highest frequency-weighted rms value. For the floor and forklift types here, typical such values were evaluated to somewhere between 0.02 and 0.05 $(m/s^2)_{rms}$. They may be compared to the limiting value of 0.04 $(m/s^2)_{rms}$ recommended for 'workshops' given in [5]. The result of such a comparison is basically in agreement with the practical experience here; Vibrations of this character and magnitude usually seem to be acceptable to the personnel involved in goods handling, but would be annoying to someone in an office-like environment.

4. CONCLUSIONS AND RESEARCH NEEDS

The experience gained from the operation of Swedish post terminals is here combined with some results from research and some specific experimental studies carried out in a couple of these terminals. The result is presented in a condensed form.

Building areas for different purposes such as goods handling and offices should be structurally decoupled as much as possible. This may be achieved by the use of moment-free joints between adjacent floor spans and by avoiding the use of non-supported partitions of full storey height, which could transfer vibration from a dynamically loaded floor to a floor supporting more quiet activities, e.g. an office.

Mezzanine decks should not be suspended from floor spans which experience dynamic service loads. Measurements at Årsta post terminal confirmed this for a case where an office floor is suspended from the floor above, which was designed to withstand forklift traffic.

Floor surfaces should be smooth and in level if they are supposed to carry traffic loads. Methods for the specification, execution and verification of such high quality surfaces are needed and further research is welcome in this area.

It is of great value to be able to compare subjective judgements, vibration measurements and theoretical calculations for floors of different construction supporting similar type of activities. Such comparisons have been made and will be continued aiming at simplified design methods for vibration serviceability.

Floors with a relatively high bending rigidity in a direction perpendicular to the span direction have better dynamic properties than floors with a more pronounced anisotropy. For concrete floor elements with a TT-shaped cross section, improvement can be achieved by adding a structurally effective (at service load level) concrete topping.

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