Free discussion

Objekttyp: **Group**

Zeitschrift: **IABSE reports of the working commissions = Rapports des commissions de travail AIPC = IVBH Berichte der Arbeitskommissionen**

Band (Jahr): **4 (1969)**

PDF erstellt am: **22.09.2024**

Nutzungsbedingungen

Die ETH-Bibliothek ist Anbieterin der digitalisierten Zeitschriften. Sie besitzt keine Urheberrechte an den Inhalten der Zeitschriften. Die Rechte liegen in der Regel bei den Herausgebern.
Die auf der Plattform e-periodica veröffentlichten Dokumente stehen für nicht-kommerzielle Zwecke in Lehre und Forschung sowie für die private Nutzung frei zur Verfügung. Einzelne Dateien oder Ausdrucke aus diesem Angebot können zusammen mit diesen Nutzungsbedingungen und den korrekten Herkunftsbezeichnungen weitergegeben werden.
Das Veröffentlichen von Bildern in Print- und Online-Publikationen ist nur mit vorheriger Genehmigung der Rechteinhaber erlaubt. Die systematische Speicherung von Teilen des elektronischen Angebots auf anderen Servern bedarf ebenfalls des schriftlichen Einverständnisses der Rechteinhaber.

Haftungsausschluss

Alle Angaben erfolgen ohne Gewähr für Vollständigkeit oder Richtigkeit. Es wird keine Haftung übernommen für Schäden durch die Verwendung von Informationen aus diesem Online-Angebot oder durch das Fehlen von Informationen. Dies gilt auch für Inhalte Dritter, die über dieses Angebot zugänglich sind.

Ein Dienst der *ETH-Bibliothek*
ETH Zürich, Rämistrasse 101, 8092 Zürich, Schweiz, www.library.ethz.ch

http://www.e-periodica.ch

DISCUSSION LIBRE / FREIE DISKUSSION / FREE DISCUSSION

Statistical Distribution of Axle-Loads and Stresses in Steel Railway Bridges

LADISLAV FRÝBA
Doc.Ing.Dr.Sc.
Chief Research Scientist
Transport Research Institute
Prague, Czechoslovakia

The paper presented by the author at the 8th Congress of IABSE in New York dealt with the non-stationary random vibration of a beam loaded by a moving random load (1). This theoretical paper has found applications in connection with bridge problems and it has, therefore, been supplemented by some experimental research work carried out on three steel railway bridges.

Here, I would like to explain some of the fundamental results and to discuss some of the experimental observations.

First of all, the instantaneous, i.e., the static and dynamic, axle loads were measured and the results were evaluated by means of histograms and distribution functions. It was determined that the mean value of the instantaneous axle load is 13,1 Mp. This is much lower than the Czechoslovak Building Code value, which is 24 Mp. The root-mean-square deviation, 5,4 Mp, is, on the contrary, very high. The speeds of the trains and the number of axles were also evaluated from the statistical point of view.

The stresses in the main structural members of the bridges, i. e., in the bridge girders and in the cross- and longitudinal beams, were also measured under the usual service load. The stresses were classified with respect to the transient time and to the crossings of 50 kp/cm^2 thresholds. The number of cycles of stresses was also evaluated by means of histograms and distribution functions. We determined that the mean stresses in all the main bridge members under traffic load were approximately 200 to 300 kp/cm^2. These average values are much lower than the standard code values, which are about 1000 kp/cm^2 in this particular case. However, the root-mean-square deviations of the stresses reach very high values, up to 200 kp/cm^2, and they are caused more by the statistical deviations of the static axle loads than by their dynamic effects. Moreover, we attempted to compare the series of local maximum stresses with the corresponding series of axle loads, but the results so far are not satisfactory.

We also attempted to measure and to evaluate the higher statistical and probabilistic functions necessary for the random vibration concept, i.e., the correlation functions and the spectral densities of variation of the stresses in some bridge members. The results, however, appeared to be extremely heterogeneous, so that hitherto no conclusions could be drawn from them and further research work seems to be necessary.

Reference

(1) L. Frýba: Non-Stationary Vibrations of Bridges Under Random Moving Load. 8th Congress of IABSE, New York, 1968, Theme VI 11

III

Effets du vent sur les constructions

D. SFINTESCO
France

Le remarquable rapport de C.W. Newberry, traitant d'un sujet des plus importants et actuels pour la construction aurait pu faire l'objet d'amples et intéressantes discussions. Il est donc regrettable qu'il n'ait donné lieu à aucune intervention, préparée ou non. Une raison de cette carence réside peut-être dans le fait que cet exposé technique n'aborde pas l'aspect probabiliste de la question, aspect essentiellement lié au thème général du Symposium.

Il me paraît indispensable de souligner l'importance de cet aspect, puisque, tant que les sollicitations extérieures - et notamment celles dues à des phénomènes aussi aléatoires que le vent - n'auront pas été définies dans le sens probabiliste, l'évaluation du degré de sécurité des ouvrages reste illusoire. En effet, la plupart des réglements actuels imposent, pour les vents dits "normaux" ou exceptionnels", des valeurs plus ou moins arbitraires, parfois modifiées au hasard des conclusions tirées d'un évènement spectaculaire local. Or, il faudrait que ces valeurs puissent être assorties d'indications sur leur probabilité - ce qui implique la nécessité de disposer de données statistiques suffisantes - et que cette probabilité soit normalisée sur le plan international, afin de rendre comparables les règles pour le calcul des constructions dans les divers pays.

Le rapport présenté constitue une excellente synthèse des connaissances actuelles dans le cadre traité. Je suis donc d'accord sur son contenu, mais je ne le suis pas pour autant sur son titre.

En effet, le problème de la sécurité des constructeurs vis-à-vis du vent présente deux parties distinctes : l'action du vent et les effets de cette action, c'est-à-dire le comportement de la structure. Or, le rapport traite de la première partie et non de la seconde. J'estime donc qu'il devrait être intitulé en conséquence.

Il ne s'agit pas là d'une simple querelle de mots, car à travers une telle imprécision de terminologie on risque de faire passer au second plan, voire de faire oublier, le deuxième volet du problème, qui est celui qui importe en fin de compte, en tant qu'élément essentiel de la sécurité de l'ouvrage, les sollicitations extérieures n'étant que les données de base pour l'étude du problème.

On peut d'ailleurs remarquer que les moyens d'investigation mentionnés pour l'action du vent ne sont pas tous applicables pour déterminer la réponse de la structure. Ainsi, les études en soufflerie ne sont d'aucun secours dans ce domaine, car on ne peut pas réaliser, à l'échelle d'un modèle réduit, la réplique fidèle du comportement très complexe d'un bâtiment complet. Les études sur bâtiments réels - coûteuses et difficiles à interpréter, mais qui finalement devraient être plus révélatrices - n'en sont qu'à leur début. Actuellement, on est donc limité aux études théoriques sur modèle mathématique, d'une valeur scientifique certaine, mais fondées sur des hypothèses simplifiées et plus ou moins arbitraires. On n'a donc pas la garantie de serrer la réalité d'assez près.

On est ainsiobligé de reconnaître l'insuffisance de nos connaissances, notamment sur les effets du vent dans la structure des bâtiments à étages, ce qui conduit à prendre des marges de sécurité probablement excessives dans les calculs. L'équilibre que l'on doit rechercher, entre les impératifs de la sécurité et de l'économie, s'en trouve compromis.

° °
°

A propos de l'aperçu historique donné dans le rapport, je voudrais remarquer indicemment que, à côté de Sir B. Baker, Irminger et Stanton, il convient de citer Gustave Eiffel, pionnier des études aérodynamiques et analyste clairvoyant du comportement des structures, dont les publications revêtent aujourd'hui encore un caractère d'actualité.

III

La protection antisismique des structures

PANAIT MAZILU
Prof. Ing.
Institut des Constructions
Bucarest, Roumanie

Le problème essentiel dans ce genre de sollicitation réside dans la possibilité d'une structure d'absorber par sa déformation l'énergie cinétique imprimée par le séisme. La réserve d'énergie de déformation dont peut disposer une construction par sa déformation au delà de la limite élastique, dans le domaine plastique, ne peut et ne doit pas, en principe, être négligée. Il y a, d'ailleurs, une littérature technique sur ce sujet.

Evidemment, il y a encore beaucoup de difficultés pour élaborer une théorie complète énergétique, à cause, entre autres, de la différence qui existe, d'un côté, entre le procédé global d'évaluer l'énergie de déformation d'une structure et, d'autre côté, le caractère local de la rupture qui peut entraîner la ruine totale de la construction.

Mais il ne faut pas ignorer l'existence de cette énergie de déformation plastique, dans les conditions d'une construction rationnellement conçue.

La manière de traiter l'action d'un tremblement de terre à l'aide des forces sismiques est certainement conventionnelle. On ne doit pas oublier qu'en réalité ces forces n'existent pas comme des forces extérieures; il s'agit en réalité d'une énergie cinétique qui peut être absorbée par l'énergie de déformation plastique , du moins dans un cas extrême d'une sollicitation sismique, toujours possible, supérieure à celle prévue par les normes officielles et les données statistiques de la probabilité des sollicitations défavorables.

C'est pour cela que le problème de la sécurité des structures dans les régions sismiques doit être traité d'une manière un peu différente.

Il s'agit non seulement des considérations théoriques, mais aussi des conséquences pratiques pour l'élaboration des projets.

A titre d'exemple, à Bucarest, pour la construction d'un hôtel en béton armé de 80 m de hauteur, ayant un plan compliqué, on a prévu des ouvertures étroites de grande hauteur, sans béton, pour réduire la rigidité excessive des diaphragmes verticaux, mais avec une armature d'acier doux, capable de supporter des déformations plastiques et, par conséquent, d'accumuler une importante énergie de déformation.

Some remarks concerning the introduction of Mr. G.R. Mitchell on "loading on buildings"

F.K. LIGTENBERG
Holland

1) Exceptional loads occur in many cases only during a very short time (a few hours). Examples are moving of furniture, people gathering for a reception, fire, repair activities. It seems very improbable that these loads will be found in an inspection in a certain number of office blocks or something like that. This means, that the imagination of the man who does the research (or of the designer) must be directed towards visualising exceptional circumstances. Much can be learned from case studies where something has gone wrong.

2) It seems a good idea to control circumstances in some way. This would mean however, that we as structural engineers have the task to give good information (understandable for people without technical knowledge) to the users of a building. For a washing machine this is quite conventional, for a building not!

3) Partitions form a considerable part of dead weight loads. We ought not to introduce these as "uniform" loads without taking account of their structural action.

4) Point loads are very important for the right dimensionering of details of a structure (e.g. holes in floors). An "equivalent" uniform load on a small floor area is a point load.

Free Discussion / Discussion libre / Freie Diskussion

C. ALLIN CORNELL
M.I.T.
Cambridge, Mass.

I should like to make several random comments on loadings based on recent research and experience.

1. Load Studies; Load studies are expensive and no little care should be taken to avoid collecting more information than is needed. It is important to remember that the interest is not in the data for its own sake, but for eventual use in guiding structural design. This simple observation has led to several data collection implications. For example,

a. If one is satisfied with estimating the member forces in supporting beams and columns it appears to be satisfactory to obtain rather gross information about the spatial disposition of the loads. Analysis suggests that the U.S. National Bureau of Standards scheme of recording the load location as simply being in one of nine sections within a room introduces negligible uncertainty in the member force prediction.

b. The load data uses seem to dictate a need for either extreme load data or simple means and variances, but not for complete descriptions.

i) For design of slabs and members sensitive to "local" loads, data from the upper tail of the load probability distribution is needed. This can probably be obtained most cheaply by training crews to sample "conditionally," e.g., with orders to measure only rooms which they estimate by quick visual check to have loads in excess of x pounds per square foot.

ii) For design of members with respect to non-failure limit states (e.g., deformation, cracking, etc.) and for members, such as major columns and footings, which support the sum of many room or bay loads, it appears to be satisfactory to estimate only the mean and variance. Sufficiently accurate estimates can be obtained with only ten to twenty rooms per building (or perhaps per firm.) Obtaining estimates of the building-to-building variation is very important, if, as some suspect, this variation is significant compared to within-building variation. The reason will be demonstrated below.

c. The degree of spatial correlation among loads in a building is important in major members, such as columns, which support many individual loads. If a column supports two floor loads (with common variance σ^2) the variance of the column load is $2\sigma^2(1+\rho)$, in which ρ is the correlation coefficient between the loads. Since ρ is probably positive in this case, the estimate of the column variance can be underestimated by a factor as large as 2 if the common simplification of independence ($\rho = 0$) is adopted.

d. A primary source of this spatial correlation can be among or building-to-building variation. A discussion by R.B. Corotis and me (in the July 1969 Journal of the Structural Division of ASCE) shows that, even if there is no within-building spatial correlation, the correlation coefficient between the two floor loads is $\rho = \sigma_A^2/(\sigma_A^2 + \sigma_w^2)$ = (among)/(among + within). Clearly this number will be significantly larger if among-building variation is large compared to within-building variation. This conclusion supports the need for adequate sampling of many different buildings, not simply careful sampling within buildings.

e. As others have mentioned these loads, being measured as they are, at effectively random points in time, do not represent observations of the maximum peak loads during a building's lifetime. Mr. Mitchell's suggestion of treating occupancy changes as being randomly selected from the (spatially measured) population seems quite reasonable. For smaller members, at least, some consideration must be given also to loads due to concentrations of people. The N.B.S. is recording open, unloaded area as a simple measure of the potential for loads due to people. Rooms heavily loaded with static loads can be expected to have less potential for loads due to people, i.e., a negative correlation can be expected between static load and unloaded area (or "people-load potential").

2. Load Combinations; The problem of properly combining loads in probabilisticly based codes has been referred to here several times. It is important to keep in mind in this regard that loads (or load effects) are in fact random functions of time. A variety of random variables associated with such random functions are important. When designing for peak gravity loads, the designer should be interested in the mean, variance, characteristic value, etc., of the random variable: peak live load during the structural lifetime. Design for wind combined with live load is another problem, however. As many have observed, it is unlikely that the peak lifetime wind load will occur simultaneously with the peak live load. Comparing the rapid versus slow fluctuation of the two random time functions and assuming that they are effectively uncorrelated functions, it would appear to be reasonable and practical to treat this combined loading by adding to the peak wind load random variable, the instantaneous (i.e., arbitrary point in time) live load random variable. This is, of course, precisely the variable which is being observed in the present load surveys.

3. Earthquake Risks; To support a previous discussion that illustrated that probabilistic methods are to be used in design, I can cite recent experience in using probabilistic methods (Cornell, B. Seis. Soc. Amer., V. 54, No. 5) to estimate the probability of exceeding design earthquake intensity values for nuclear power plants. Interestingly enough, when several different sites were analyzed in this manner and the probabilities calculated for

each of the two rather arbitrarily defined design levels ("maximum probable" and "maximum credible"), both of which had been previously, independently selected by rather arbitrary means, a surprising degree of consistency was found.[3] The former level usually corresponded to a return period of about 10^3 years and the second to 10^5 or 10^6 years.

III

Free Discussion / Discussion libre / Freie Diskussion

B.E. WEINBERG
U.S.A.

I would like to add the following comments to Mr. Newberry's fine paper:

1. I wholeheartedly agree with Mr. Newberry's plea that wind-load requirements should not be lowered until more research in this area has been completed. In the past, buildings have been far stronger in resisting wind pressures than those for which they were designed; primarily, on account of the existence of non-load bearing partitions. However, the tendency today, at least in the United States, is for office buildings to be built with moveable partitions. Many partitions that are not moveable do not extend all the way from floor to ceiling. Therefore, we no longer are guaranteed the built-in added safety factor so frequently present in the past.

2. In ACI Committee 348 (Structural Safety), we consider serviceability to be one aspect of structural safety. Therefore, it is not enough to design a building to withstand wind pressure so that the building will not collapse. The building must also be comfortable for those inside it. This gets to be important as more of our tall buildings are apartment buildings, not only office buildings as in the past. Wind deflections which might be acceptable to workers in an office building, may be totally unacceptable to tenants living in an apartment building.

Concerning Mr. Mitchell's paper, I would like to add the following comments:

1. There is usually very little control of construction loads by the designing engineer and sometimes not even by the contractor. This is a problem which engineers should consider during their design and contractors in planning their construction sequence. Many more buildings collapse during construction than after they are completed. This is especially true of concrete buildings where frequently construction loads far in excess of the design live load are imposed on parts of the structure which have not yet attained their design strength and are not intended to for twenty-eight days.

2. For snow loads, the duration of the load must be considered together with the intensity of the load.

3. In addition to those mentioned there are two other load surveys being conducted in the United States; one by the Post Office Department of its facilities and the other by the National Bureau of Standards, the latter being confined to office buildings.