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Dynamic Loads

Dynamic Loads
(In Particular Wind and Earthquake Loads)

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1. Introduction

Progress in the structural field implies the improvement of knowledge of both the acting loads and the structural behaviour under these loads. In general it can be said that the actual knowledge of dynamic loads is not great. So, it is easy to understand the interest of not only obtaining data that allow a better definition of these loads, but also of establishing the most convenient methodology for their analysis.

For the distinction between static and dynamic loads a first difficulty arises from the fact that this distinction cannot be established on the basis of the loads themselves but has to consider the type of structure on which the loads act. In fact, a load has to be considered as dynamic when its variability in time is such that, to study the structural behaviour, the effects of the inertia of the masses of the structure cannot be neglected.

A second difficulty concerning the definition of dynamic loads arises from the interaction between the structural behaviour and the loads themselves. Considering the dynamic loads as a system of forces variable in time, these forces are often directly influenced by the behaviour of the structure. This for instance occurs in wind actions due to aero-elastic effects and in earthquake problems owing to the interaction between the structure and the soil. In the present report it is tried to define the loads in a fundamental way, in order that these interactions may be studied by considering the total behaviour of the overall system.

To limit the scope of the report attention is focussed on the loads that act on civil engineering structures such as buildings, bridges and towers.

The types of loads to be referred are those due to: wind, earthquake, traffic, machinery and blast. The two first types of loads, which in general are more important, are studied in more detail.

For each type of load the main problems to be considered are: i) measurement of the acting loads, ii) their analytical and experimental representations. The discussion of the most convenient methods for dealing with these problems is very important and, as referred, it is hoped that during the Congress the discussion shall cover methodology and not be limited to the presentation of results.

For the purpose of choice of method two fundamental types of representation are considered: deterministic and stochastic. The deterministic representation implies the complete knowledge of the variation of the loads with time. The stochastic representation only implies the knowledge of the statistical distributions of these variations.

Machine vibrations or blast pressures may be examples of loads for which a deterministic approach applies. Wind and earthquake loads in general can only be conveniently represented by stochastic schemes.

The improvement of load knowledge implies that systematic measurements are performed in nature. The dynamic character of the loads makes these measurements particularly difficult. The difficulties are of two principal kinds. The first comes from the fact that often it is not possible or practical to measure directly the acting forces but only other magnitudes from which the forces are to be derived. That is for instance the case for the wind for which it is the velocity and not the pressure that has to be considered. So a new problem has to be solved: to transform the velocities in pressures.

The second difficulty comes from the response of the measuring device in function of the frequencies involved. In fact the measuring devices behave in general as low-pass filters, have often a non-flat frequency response and even in some cases are non-linear, that is, the response is a function of the mean intensity of the load. For a correct interpretation of the results, the measuring devices have to be considered as transfer systems and the characteristics of their response accurately studied.

Difficulties of this type appear for instance in relation to the use of anemometers. The results available have to be interpreted by duly taking into account the dynamic characteristics of the apparatus.

Once the nature of the loads is well understood the convenient representations of the loads to be used in analytical or experimental studies have to be discussed. The representations must be of a nature as fundamental as possible in order that they can be applied with generality. Also the analytical representation of the loads must fit the general theories to be used for studying structural behaviour. Particularly in the stochastic scheme the fundamental concepts of the theory of random vibrations [1, 2] must be respected.

It is also convenient that analytical representations are as simple as possible. The influence of the introduced simplifications must be analysed and the range in which they apply must be defined.

In the case of permanent loads the definition of duration is not important. In the case of non-permanent loads it is convenient to split them up by the consideration of intervals with given durations. So wind and earthquake loads shall be represented by time series having each a given duration. A further usual simplification consists in supposing that the statistical distribution from which the time series derives does not change along the time for the assumed duration; that is to suppose that the phenomena are stationary.

The experimental representation of the dynamic loads must also satisfy some general conditions. Loads representing the actual ones are to be imposed on models and the behaviour of these models has to be interpreted using theories of similitude. The frame in which similitude applies imposes conditions on the load representation. When interpreting the results the limits within which the tests were performed have to be duly considered.

So, for instance, wind tunnel tests in general disregard wind gradients and do not respect the similitude for turbulence. Dynamic tests for the study of earthquake problems often use vibrations that are far from representing earthquake movement.

To allow a comparison between analytical and experimental results it is convenient that analytical and experimental representations of the loads are in accordance.

Finally, the problem of load forecasting has to be considered. This forecasting must in general be established on statistical bases. Statistical concepts of safety may then be applied.

It must be well understood that this statistical forecasting has nothing to do with the (deterministic or stochastic) scheme adopted for the representation of the load variations in time.

The general problems concerning dynamic loads on structures have been recently studied in several symposia, among which the "RILEM Symposium on the Measurement of Dynamic Effects and Vibrations of Constructions" held in Budapest in 1963, the "Symposium on Vibration in Civil Engineering" held in London in 1965 [3], and the "RILEM Symposium on the Effects of Repeated Loading of Materials and Structures" held in Mexico in 1966. The particular problems concerning wind and earthquake loads have been dealt with in special meetings that are referred below. Davenport in the report presented at the symposium held in Mexico [4] makes an interesting general analysis and a comparison between wind and earthquake loads.

2. Wind Loads

2.1. Nature of wind

A good understanding of wind phenomena implies the general knowledge of wind causes.

As is well known, the wind velocity increases with the height above ground and for heights of about 300 to 600 m reach a limit value (gradient velocity) that is mainly governed by pressure gradients (direct connected with thermal effects) and geostrophic accelerations. This gradient velocity can be analytically related to the mentioned causes.

The wind velocity rapidly varies with time and from point to point. This variation being random the flow is turbulent.

For the study of wind loads it is convenient to consider intervals of time with a given duration (for instance 10 minutes or 1 hour) and to compute for these time intervals the correspondent mean wind velocity. The rapidly varying velocity component then corresponds to turbulence.

Recent studies on the structure of wind that duly consider turbulence have been principally performed in connection with air pollution [5] and aeronautics [6]. Much information coming from these sources is now available but only a small part can be used for studying the wind action on structures. Even so, it must be recognized that the methodology used in these studies is the convenient one. Modern studies on the wind action on structures follow the same lines [7, 8].

2.2. Wind measurement

The measurement of the wind velocity may be divided into two different problems. The first concerns the measurement of the low frequency component that corresponds to the mean velocity. The second concerns the high frequency component due to turbulence.

Practically all over the world the measurement of mean wind velocities has been in charge of the meteorological services and a large information is available on them. On the contrary measurements of turbulence have been performed in relation with particular researches only.

The meteorological services also indicate maximum velocities but these are difficult to interpret and for the time being are unreliable for use in structural design.

The different types of anemometers for measuring wind velocity can be classified according to their range of frequency.

Among the low frequency types are: Pitot static tubes, propeller and vane anemometers. These instruments have in general cut-off frequencies of about 0.1 c/s.

In the medium range of frequency are membrane or vibrating mass anemometers with cut-off frequencies of the order of 20 c/s.

Finally in the high range of frequency can be considered the hot wire and electric discharge anemometers that respond up to frequencies of about 1000 c/s.

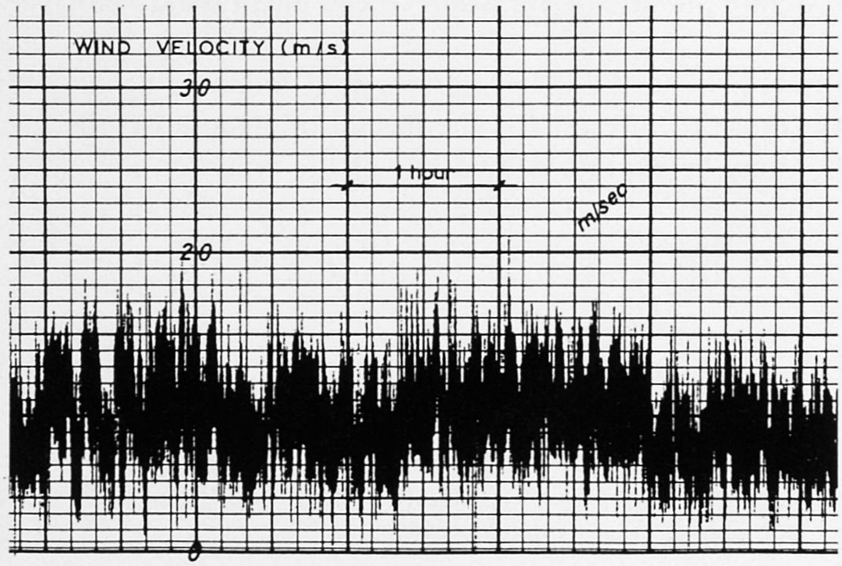

Fig. 1. Typical record of wind velocity

Fig. 1 presents a typical record obtained using a Pitot tube anemometer.

For the interpretation of the records of these different types of anemometers it is necessary not only to calibrate them statically in wind tunnels but also to determine their transfer functions. Besides, their behaviour being often not perfectly linear, the transfer functions have to be determined for different values of the static component.

Measurements in turbulent flows, and particularly the measurement of turbulence itself, involve important difficulties [9]. These difficulties are much reduced if only mean wind velocities are considered. Owing to the presented reasons, only the mean values of wind velocity constitute a set of valuable data from which the maximum velocities can be derived. The relashionship between mean and maximum velocities has to be based on the actual knowledge of the structure of wind. The spreading of the correct measurement in the high frequency range is most desirable. The data then obtained shall allow not only a better understanding of turbulence but also direct estimates of maximum velocities.

2.3. Representation of wind

2.3.1. Mean wind velocity

For the study of the wind loads on a structure it only interests to consider a limited spatial domain surrounding the structure. Also, as referred, only a given interval of time has to be considered, for instance 10 minutes or 1 hour.

In general it shall be reasonable to suppose that in the given domain the mean velocity is represented by a horizontal component that only varies in function of the height. To define the field of mean wind velocities it is then necessary to define the law of variation of the mean wind velocity with height only.

For the range of height that interests civil engineering and for winds of high velocity (those of interest for the design of structures) the mean velocity at level z, $\bar{U}_z$, may be expressed by a law of the type

$$\bar{U}_z = \left(\frac{z}{z_g}\right)^{1/\alpha} \bar{U}_g, \tag{1}$$

where $\bar{U}_g$ is the gradient velocity at level z_g. The values of z_g and $1/\alpha$ depend on the roughness of the ground.

As the wind velocities are in general measured at heights of about 10 m above the ground it is convenient to use for reference the velocity at this height, $\bar{U}_{10}$, and not the gradient velocity.

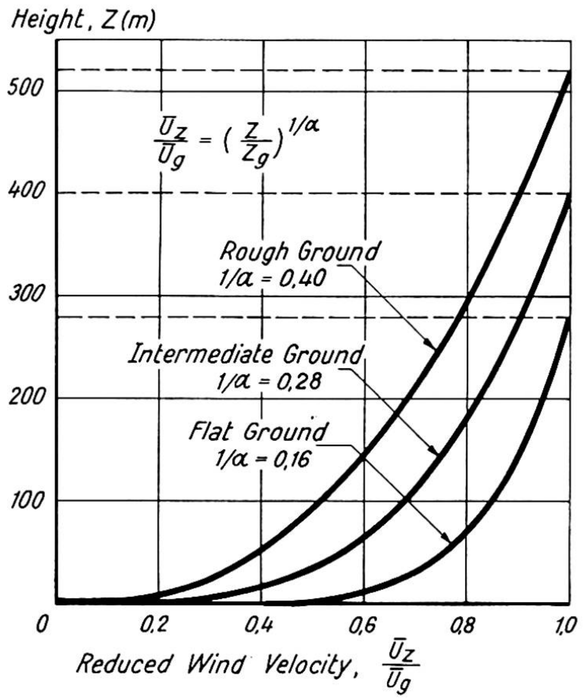

Fig. 2. Mean wind velocity profiles according to Davenport [8]

Fig. 2 gives the mean wind velocity profiles, according to Davenport [8], for three typical conditions of ground roughness.

To each location there corresponds a statistical distribution of $\bar{U}_{10}$, that defines the probability of this velocity being exceed during a given interval of time.

2.3.2. Turbulence of wind

Due to the existence of turbulence it is necessary to add to the mean velocity a varying velocity defined by the components u, v, w (respectively longitudinal, transverse and vertical).

These components are supposed to have a random variation in time and space. The statistical definition of these variations can be performed only by assuming simplifying hypotheses.

A first reasonable hypothesis consists in supposing that the turbulence is stationary during the time interval considered. The variation of velocity at a point may then be statistically described by the variation of the spectral densities of the velocities in function of the frequency or by the Fourier transform of these spectral densities, the auto-correlations. Assuming homogeneity at the different levels, the spectral densities of velocity may then vary in function of the mean wind velocity and the height above ground.

Spectral densities in time are not sufficient to define turbulence completely. It is also necessary to consider the statistical variation in space described by the space spectral densities or space correlations.

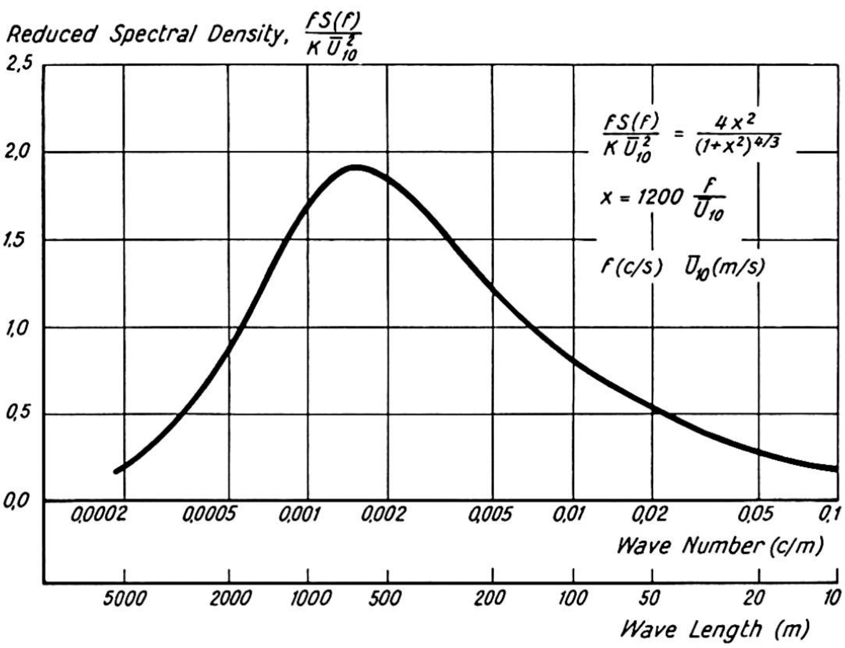

Fig. 3. Spectral density of longitudinal component of wind velocity according do Davenport [8]

For aeronautical problems the turbulence of the vertical component is very important [6]. The same does not apply, in general, to the wind actions on structures. In this case the longitudinal component is of particular interest.

Fig. 3 presents the spectral density of the longitudinal component of wind velocity proposed by Davenport [8]. It represents the mean value of measurements performed at heights ranging from 8 to 150 m. As means were taken, this implies to consider that turbulence does not vary with height in the range considered.

For representing the spectra obtained in different conditions by a single curve, Davenport divides the spectral density $S(f)$ by a coefficient K, a surface drag coefficient depending on the ground roughness, and by $\bar{U}_{10}^2$ the square value of the mean velocity at the reference level. The abscissae represent in a logarithmic scale not directly the frequencies but wave numbers or wave lengths. The wave number, expressed in cycles per meter, may be reduced to the frequency (c/s) by multiplying it by the mean velocity (m/s). The wave length is obtained by dividing the mean velocity by the frequency.

Taylor's hypothesis consists in supposing the validity of the transformation $x = \bar{U}t$, that is, it establishes an equivalence between the variations in space (x) and in time (t). According to this hypothesis the presented spectral density in time may also be considered as a spectral density in the longitudinal direction.

As the abscissae are represented in a logarithmic scale it is convenient to multiply the ordinates by the frequency f in order that the integral of the spectral density represents the mean square value of the velocity fluctuations.

For wave lengths smaller than 500 m the proposed spectrum fits Kolmogorov's law [10]. This law relates the spectral density $S(\lambda)$ to the wave number, λ, by an expression of the type $S(\lambda) = \beta \lambda^{-5/3}$.

To get quantitative information that may be used in structural design the spectrum of fig. 3 is plotted in fig. 4 for a mean velocity of wind of 20 m/s.

This figure shows that the reduced spectral density, $fS(f)/\sigma^2$, has a maximum between 1 and 2 cycles/minute. As the frequency increases the spectral density rapidly decreases and for practical purposes may be considered to vanish above 1 or 2 c/s.

The relative intensity of turbulence that corresponds to the spectra of fig. 3 and 4 is given by $\sigma/\bar{U}_{10} = \sqrt{6K}$. For $K = 0.005$ this corresponds to a value of 0.17 that is in accordance with the usual intensities of turbulence, between 10 and 20%, indicated by PASQUILL [10].

The variation of the longitudinal component of wind velocity in transverse directions can only be defined by considering the correlation in these directions or the cross spectral densities $S(\Delta l, f)$ for points at different distances, Δl.

DAVENPORT [11] relates the cross spectral densities to the spectral density in time, $S(f)$, by means of a coefficient

$$R(\Delta l, f) = \frac{S(\Delta l, f)}{S(f)} = e^{-\frac{cf\Delta l}{\bar{U}_{10}}},$$

where c is a constant. $\bar{U}_{10}/cf$ has the dimensions of a length and can be interpreted as a correlation scale.

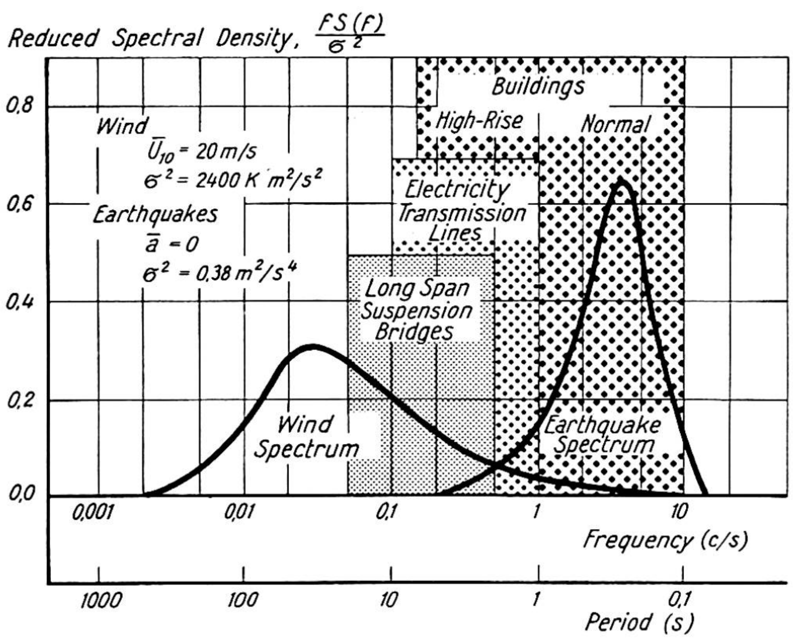

Fig. 4. Reduced spectral densities of wind velocity and earthquake acceleration

The distance at which wind velocities are correlated is therefore inversely proportional to the frequency and to the coefficient c. Table I indicates for, stable atmospheric conditions (that are those of interest in the present case), the order of magnitude of the value of c for different directions and turbulence components [11].

Table I

Direction of Δl	Turbulence component	c
longitudinal	longitudinal	8
	transverse	6
transverse	longitudinal	40
	transverse	25
vertical	longitudinal	7
	transverse	7

The fact of the correlation scales being smaller in the transverse than in the longitudinal direction indicates that for strong winds eddies are elongated.

2.4. *Wind actions*

The knowledge of wind velocities is not sufficient to define wind actions. It is necessary to know how to transform the velocities in pressures. Duly considering the dynamic character of the phenomena this problem is not yet satisfactorily solved.

Supposing the wind to be a uniform flow, much information is available that allows to transform the wind velocity into local or total pressures. This information is collected in wind codes under the form of pressure coefficients.

JENSEN [12] has shown that if the uniform flow is substituted by a boundary layer profile, as indicated in fig. 2, important variations of the pressure coefficients are obtained.

Considering the turbulent character of wind the problem is much more involved. In fact to the turbulence inherent to the wind it is necessary to add the turbulence created by the structure.

The turbulence created by the structure derives mainly from vortex excitation and may be independent of or dependent on the deformability of the structure itself.

The first type of phenomena corresponds to von Kármán eddies. The frequency, f, of the vortex-shedding is related to the mean wind velocity, U, by the Strouhal number, $S = fD/\bar{U}$, where D is a typical length. As there is a dominant frequency f this is not a true turbulence; it is often called a quasi-turbulence. A review of the problem of vortex-shedding for rigid circular cylinders is presented by LIENHARD [13].

The excitation of vibration due to the deformability of the structures is a still more complex phenomenon. A simple case where the mechanism of the vibrations can be easily understood is the following.

Consider a horizontal wind and a structure vibrating vertically. The vibration of the structure corresponds to a transverse component of velocity. To combine this transverse component with the longitudinal one is equivalent to consider an oblique incidence. If the profile of the body is such that for this oblique incidence there is a negative lift, this lift force tends to increase the vibration of the system and may be considered a negative damping. If this negative damping exceeds the positive damping of the structure, self-excited vibrations of increasing magnitude will occur.

For systems with several degrees of freedom the phenomena of aerodynamic instability may be associated with coupling of different modes giving rise to flutter phenomena.

Finally it may also occur that the turbulence produced by one structure influences neighbouring ones.

SCRUTON [14] reviews the different aspects of wind-excited vibrations of structures. Owing to the complexity of the involved phenomena it can be said that general analytical solutions cannot yet be obtained. Model tests in wind tunnels yield very useful results.

2.5. *Model tests*

Usually, when building a wind tunnel it is sought to reduce turbulence as much as possible in order to obtain a uniform air flow. A tunnel in these conditions is well adapted for determining pressure coefficients, but obviously does not allow to study the influence of the turbulence of natural wind.

Recently several tunnels were built [8] to reproduce the variation of wind velocity with height. This is obtained by increasing the roughness of the deck wall in comparison with that of the other walls. Fig. 5 represents the boundary layer wind-tunnel of the University of Western Ontario.

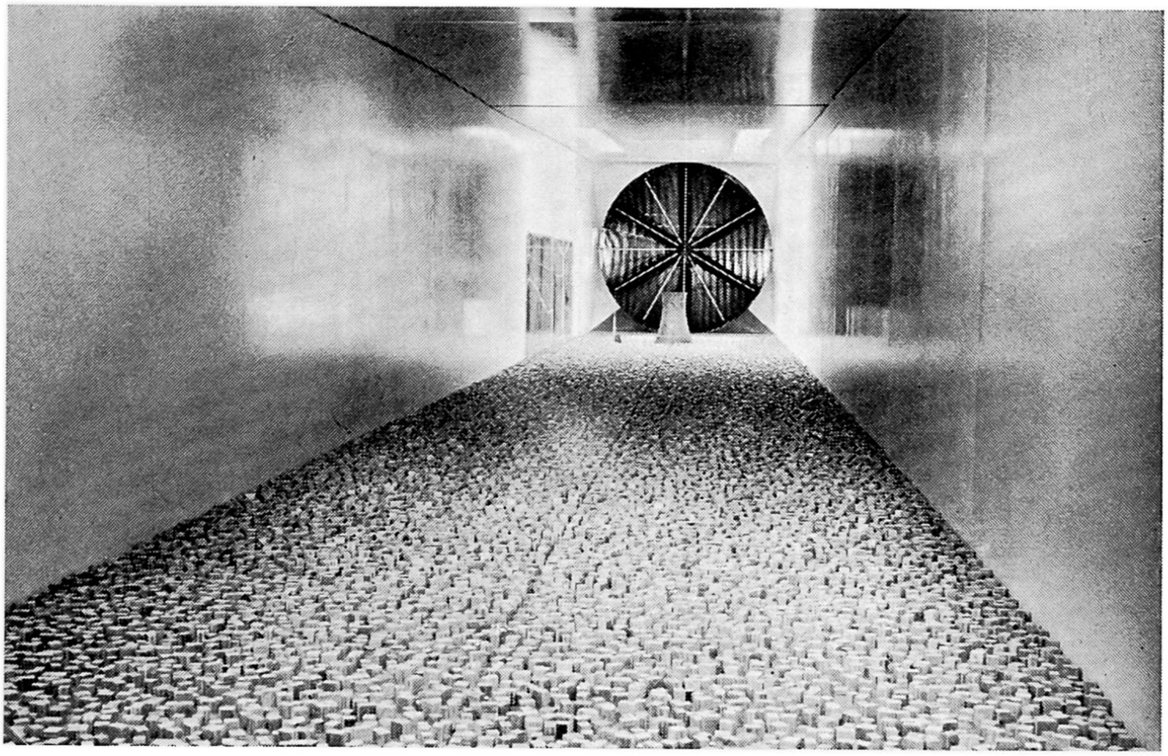

Fig. 5. Boundary-layer wind tunnel of the University of Western Ontario

In future it would be desirable to be able accurately to control the turbulence of wind tunnels in order to perform dynamic tests with the turbulence reproduced to scale.

2.6. *Data for design*

Among the data for design it is necessary to define the maximum wind velocities to which the pressure coefficients apply. The maximum velocities at present used are not currently defined on a statistical basis.

It is important to be able to relate maximum and mean velocities. Several authors studied this problem [15, 16]. CASTANHETA [17] adopting the velocity power spectrum indicated by DAVENPORT [8] computed the statistic distribution of the ratio of the maximum velocities, U_f, to the mean velocities recorded in time intervals $t_0 = 10$ minutes or 1 hour. The maximum velocities, U_f, are those that would be recorded by ideal filters with sharp cut-off frequencies $f = 0.2$, 0.5, 1.0 and 2.0 c/s.

Table II indicates the obtained ratios of the mean maximum velocities to the mean velocities. These values refer to a height of 10 m above ground.

Table II

Cut-off frequency $f(c/s)$	$t_0 = 10$ minutes			$t_0 = 1$ hour		
	Roughness coefficient, K[1])			Roughness coefficient, K[1])		
	$K=0.005$	$K=0.010$	$K=0.015$	$K=0.005$	$K=0.010$	$K=0.015$
0.2	1.44	1.63	1.77	1.54	1.75	1.92
0.5	1.50	1.71	1.87	1.58	1.83	2.02
1.0	1.53	1.75	1.92	1.63	1.88	2.07
2.0	1.56	1.79	1.97	1.66	1.92	2.13

[1]) As defined by DAVENPORT [8]. $K = 0.005$ corresponds to a flat ground and $K = 0.015$ to rough conditions, as those occurring in the centre of a town.

It is interesting to notice that the variation of the coefficients due to the variation of the cut-off frequencies is practically independent of the roughness coefficient and the time interval.

Taking for reference the frequency of 2.0 c/s, reductions of 2, 4 and 9% are obtained for the cut-off frequencies of 1.0, 0.5 and 0.2 c/s, respectively.

Assuming that a structure behaves like a filter with a given cut-off frequency, the indicated reductions could be applied to the maximum velocities used for design. As the pressures are proportional to the square of the velocities, the reductions of the pressure would be twice the indicated ones.

In the case of structures with long fronts a further reduction can be considered related to the transverse correlation of wind velocities. So far these reductions have been mainly established on experimental bases [18, 19]. The information now available on the turbulence of wind is already sufficient to allow an analytical determination of these reductions [17, 20].

The above considerations show that reductions of wind loads due to the turbulence of wind are only justified for structures of very low frequency, such as long span suspension bridges, electricity transmission lines and high-rise buildings, fig. 4. In fact structures of these types present natural frequencies of about 0.1 c/s or even less. The same does not apply to ordinary buildings (of no more than 10 stories) that in general have frequencies above 1 c/s. For structures of this type wind can be considered as a static load.

On the other hand for very deformable structures, such as suspension bridges, aeroelastic phenomena may be of paramount importance [21].

For improving the knowledge concerning wind loads it seems very important to observe the real behaviour of structures [22]. Such studies will confirm the assumed hypotheses and indicate the most promising research lines.

3. Earthquake Loads

3.1. Nature of earthquakes

It is generally accepted that earthquakes are produced by local ruptures of the earth's crust. Earthquake vibrations may be felt at very large distances from the epicentral zone but, in general, they only affect constructions in a much smaller radius. Within the area in which earthquakes are destructive, soil movements have an irregular character as can be appreciated from the available records, fig. 6.

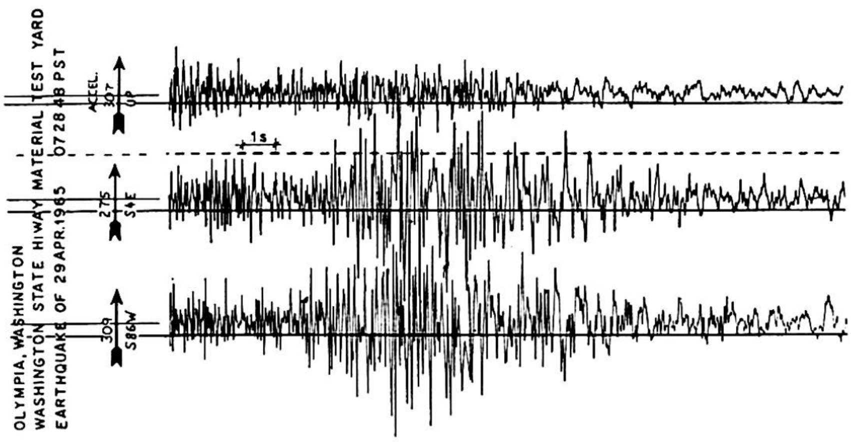

Fig. 6. Typical record of earthquake acceleration

Man-made explosions may also be the cause of soil vibrations that in some aspects may be compared to the movements due to earthquakes.

Owing to the earthquake vibrations, water masses enter in movement and may produce large waves in the sea (tsunami) and also hydrodynamic pressures on structures that are submerged in or in contact with water.

In recent years much effort has been made to investigate earthquake causes, to define the seismicity of the different regions of the globe, to study the most adequate methods to measure and represent seismic movements and to study the behaviour of constructions under earthquake loads. Among the abundant literature on these subjects, the proceedings of the World Conferences of the

International Association on Earthquake Engineering are of paramount importance [23, 24, 25]. The recent effort of UNESCO in co-ordinating and promoting research in this field must also be emphasized.

3.2. Earthquake measurement

Although the small amplitude movements of soil due to distant earthquakes have been recorded for many years, only recently has convenient equipment for recording strong motions been developed [26].

The apparatus now currently used records the three components of the soil acceleration and automatically starts the recording when the vertical or one of the horizontal components exceed about 0.01 g.

The dynamic characteristics of this equipment and the velocity at which the record is performed are adequate for further analysis of the seismic vibrations. Also the number of strong-motion accelerographs is rapidly increasing. Even so the most important seismic regions are not yet conveniently covered.

3.3. Representation of earthquakes

A very important contribution to the representation of soil movements due to earthquakes is due to HOUSNER [27] who simulated these movements by a set of random pulses and represented them by acceleration, velocity or displacement spectra. As defined by Housner, the spectrum of a given magnitude, for instance velocity, indicates the maximum values of velocity that simple oscillators with different natural frequencies and different damping undergo when subjected to the considered motion. Housner computed the spectra of several strong-motion accelerograms and proposed to represent earthquake motions by the spectra indicated in fig. 7.

Another way to describe the randomness of soil acceleration consists in defining the power spectral density of acceleration. This description uses the fundamental magnitudes of the theory of random vibrations that were also used to describe wind turbulence.

The representation adopted by HOUSNER and others [28, 29] corresponds to a white noise vibration (vibration of constant power spectral density). The variation of the power spectral density in function of the frequency was proposed by TAJIMI [30], on basis of preliminary work of KANAI [31], and by BARSTEIN [32]. In a study dealing with the probabilistic approach to earthquake-resistant design ROSENBLUETH [33] reviews the different idealizations of seismic loads.

BYCROFT [34] has shown that the velocity spectra indicated by Housner were equivalent to a white noise vibration limited to the range from 0.2 to

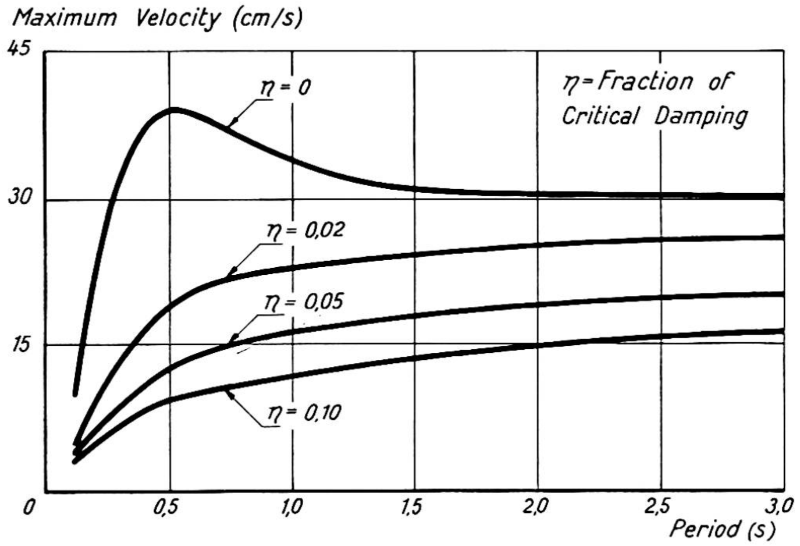

Fig. 7. Spectra of maximum velocities according to Housner [27]

5 c/s, each sample having a 30 s duration. He indicates the spectral density of 695 $cm^2\ s^{-4}/c/s$ as equivalent to the NS record of El Centro 1940 earthquake.

Recently RAVARA [35], JENNINGS [36] and ARIAS and PETIT LAURENT [37] used digital computers to determine directly from the available earthquake records the correspondent spectral densities.

Although the spectra present important fluctuations it is possible to deduce a mean law of variation of the spectral densities in function of the frequency (fig. 8). Analytical expressions for this law have been proposed by several authors [30, 36, 38, 39]. It is to be expected that the variation of the spectral density will depend on the geometry and mechanical properties of the soil. Unhappily the information now available does not yet permit to quantify this influence nor the one deriving from the epicentral distance.

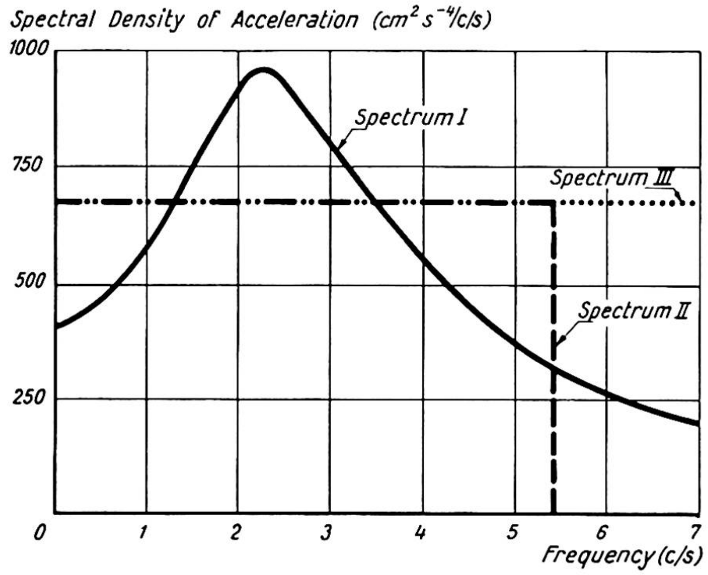

Fig. 8. Variation of the spectral density of acceleration in function of frequency

It must be emphasized that the difference between the representation by a white noise of limited range of frequencies and the spectrum function corresponding to a linear oscillator is not as important as could be imagined. In fact mechanical systems always behave as filters that cut off the frequencies above a given limit, and so it is just the same whether loading has a zero or non-zero spectral density above this limit. PEREIRA [39] compared the response of linear oscillators for 3 types of spectral functions (fig. 8) and obtained the results indicated in fig. 9 that well confirm the above conclusion.

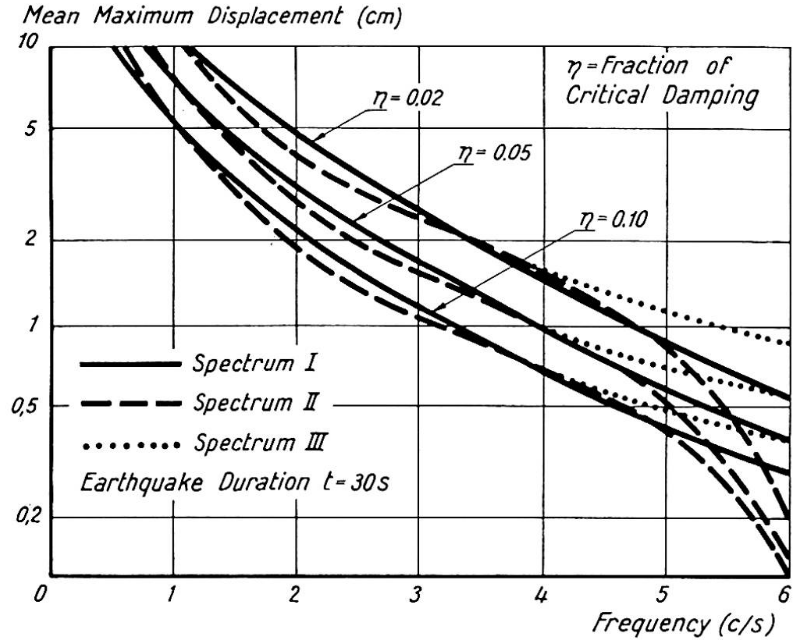

Fig. 9. Variation of the mean maximum displacement of one-degree of freedom linear oscillators in function of frequency, according to Pereira [39]

To represent the soil movements completely it would be necessary to define at each point not only the variation of acceleration with time but also the cross-correlations in different directions. There are at present no elements allowing to obtain these data. Even so the correlations in time computed by BARSTEIN [32] and ARIAS and PETIT LAURENT [37] give useful information concerning the dimensions of the areas in which the soil vibrations can be considered as approximately uniform. In fact the time interval for which the correlation falls down is of the order of 0.1 s. So for the usual velocities of propagation of the seismic waves, points at a distance of about 100 m shall simultaneously have similar movements. The same may no longer be true if the points are more than 500 m apart.

A representation of earthquakes that allows to study the non-uniformity of the vibrations at different points was proposed by BOGDANOFF, GOLDBERG and SCHIFF [40]. These authors consider packets of damped oscillatory waves with random amplitudes, frequencies, arrival times, phases and velocities of propagation. This representation is used to study the longitudinal vibration of a suspension bridge.

3.4. Structural behaviour

Analytical methods now available allow to study the behaviour of structures under very general hypotheses. Lumped-mass methods expressed in matrix form, and making use of digital computers, are a powerful tool for the dynamic analysis of structures.

The earthquake idealization presented above constitute the necessary basis for performing this dynamic analysis.

When considering earthquake representation it cannot be forgotten that, due to the interaction between the soil and the foundation, the vibrations in fact applied to the structure may be very different from those considered. This problem can only be dealt with by duly studying the behaviour of the whole structure-soil system and assuming that sufficiently far from the structure the soil vibration is in accordance with the given representations.

3.5. Model tests

Although the power of analytical methods is nowadays very much increased by the use of digital computers many problems cannot yet be conveniently solved in this way and for them model studies constitute the convenient approach. Both mechanical models and electric analogies may be considered as models, although electric analogies are in general more close to the analytical representation.

Analog computers have been used with much success to study dynamic problems concerning earthquake actions [41]. Random vibrations may be conveniently studied in this way. In fact, random noise generators constitute a standard equipment that may be used to feed the analog computer.

In dynamic tests different techniques can be followed to reproduce soil vibrations [42]. For many years sinusoidal shaking tables have been in use with control of frequency and amplitude. In other cases vibrations have been induced by impacts that produce damped sinusoidal vibrations whose frequency depends on the system of springs attached to the table.

At the Laboratório Nacional de Engenharia Civil, model tests have been mainly performed using random vibrations, since 1960 [43, 44]. Fig. 10 shows the test set-up. The loads applied to the model represent to a convenient scale the soil vibration and both the spectrum (usually considered a limited range white noise) and the duration are reproduced. By performing several tests it is possible to determine the mean maximum values of the response. By successively increasing the power spectral density it is possible to study the be-

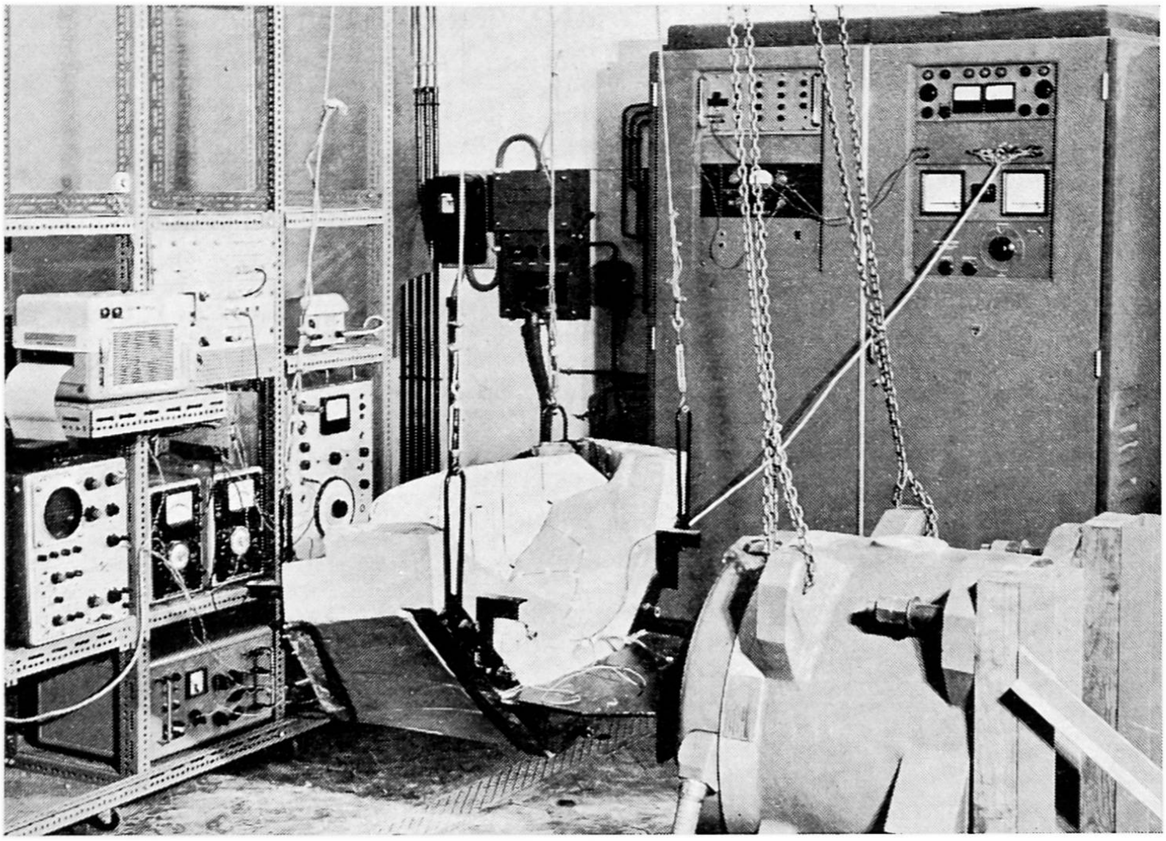

Fig. 10. Random-vibration set-up for model testing

haviour in the non-linear range and even up to rupture. Model studies in which random noise vibrations are used have also been recently performed in Japan [45, 46].

The shakers used in these studies are of the electromagnetic type and so only adequate for frequencies above 20 c/s. Automatically controlled hydraulic jack systems now produced in some countries are well adequate to perform model dynamic tests. With these jack systems forces of the order of magnitude of hundreds of tons may be applied and these forces can be varied up to frequencies of about 20 c/s [47]. As in some cases forces can be varied according to any given program, random vibrations may also be applied. It is to be expected that systems of this type shall be very usefull for seismic studies on models.

3.6. Data for design

In general the codes on earthquake resistant construction [48] specify seismic coefficients to be applied to the weight of the masses existing at the different levels, thus allowing to compute the horizontal forces for which the structure has to be designed. In this way earthquake dynamic forces are transformed in equivalent static ones.

It is very difficult to establish the seismic coefficients in order to contain all the information corresponding to a complete dynamic analysis. Although rough, the design according to seismic coefficients guarantees a resistance to horizontal forces, which is very important from a practical point of view.

Considering the difficulties of dynamic analysis, the specification of seismic coefficients may be considered a convenient procedure to give information to designers, mainly for usual types of structures as, for instance, ordinary buildings. For structures such as dams, high-rise buildings, big bridges and towers it is in general necessary to carry out a complete dynamic analysis [49].

4. Other Types of Dynamic Loads

The space available for this preliminary report only allows a very brief reference to the principal types of dynamic loads.

The following considerations are only intended to introduce and stimulate the discussion on dynamic loads besides those due to wind and earthquakes.

4.1 Traffic loads

The problem of traffic dynamic loads on bridges is entirely different according as road or railway bridges are concerned.

For road bridges the actual tendency of codes is to distinguish between loads due to traffic congestions and loads due to exceptionally heavy vehicles [50]. Intermediate hypotheses do not correspond to extreme conditions. As traffic congestions may be considered as static loads, dynamic behaviour has to be studied only for exceptionally heavy vehicles.

For railway bridges the problem is completely different. The repetition of the loads, associated very often with inversion of stresses, may produce fatigue. The study of dynamic behaviour is of paramount importance in this case.

Another type of dynamic traffic load is the one due to landing aircrafts.

For the improvement of the actual knowledge of traffic loads it seems convenient to decompose the dynamic effects into deterministic and stochastic parts. These parts would be separately analysed according to the respective theories. Thus design rules more accurate than the present ones would be obtained.

4.2. Machinery loads

Dynamic loads due to machinery may in general be established on a deterministic basis by being assimilated to periodic vibrations. In some cases, special types of machinery such as ball-mills also produce random vibrations.

Each machine vibration problem has its own pecularities and it is difficult to give general information of interest. Specialized books [51, 52] contain useful information on this subject.

4.3. Blast loads

The general term of blast refers both to soil vibrations and to fluctuations of air pressure due to man-made explosions.

Concerning soil vibrations, blast effects may be compared to seismic movements. Available records show that the accelerations may also be considered as random, but the time duration of the vibrations is much shorter than the one assumed for earthquakes [53].

Studies dealing with quarry blasting [54] relate the damage in the constructions and the acceleration or velocity peaks with the distance to the shot and the explosive charge. Other studies deal with nuclear underground explosions and analyse the records in terms of their power spectral density [55].

The air blast effects are mainly related to the explosion of nuclear weapons in the atmosphere. The pressure wave resulting from an explosion near ground consists of an abrupt rise in pressure followed by a decay from which a negative pressure half-wave results [56]. The shape of the pressure wave is well defined and so its effects on structures can be studied by using the deterministic theory of vibrations.

The value of the load to be adopted in design has to be established on a strategic basis taking into account the degree of protection that is desired.

Sonic boom can also be considered as a type of blast load.

5. Conclusions

The main purpose of this introductory report is to serve as a basis for the discussion to be held during the Congress. So the present conclusions contain proposals on research subjects about which discussion is desirable.

5.1. The need of a correct definition of the loads acting on the structures was emphasized. It is important to state this definition in a basic way according to well-established general theories. Discussion on the most convenient methodology to attain this scope is welcome.

5.2. Recent progress concerning the knowledge of the dynamic action of wind was described.

To increase the available information it seems desirable:

5.2.1. To record systematically, by convenient anemometers, the turbulence of wind, in supplement to the actual recording of mean velocities.

5.2.2. To improve the actual representation of wind by using velocity spectra that duly vary in function of geographical conditions and other pertinent variables.

5.2.3. To establish on a sound statistical basis the velocities to be used in design.

5.2.4. To study the dynamic behaviour of structures taking simultaneously in consideration the turbulence inherent to the natural wind and the turbulence created by the structure itself. For this purpose model studies considering the wind velocity profile and turbulence effects seem promising.

5.2.5. To pursue the observation of important structures in order to get more information concerning their behaviour under wind actions.

5.2.6. To include in the building codes not only simplified data for the design of usual structures, but also basic data that can serve for design of important structures.

5.3. For further progress in the definition of seismic loads the following lines seem promising:

5.3.1. To continue installing strong-motion accelerographs and duly to interpret the obtained records.

5.3.2. To define the seismic loads by the spectral density of acceleration and to consider the variations of the spectra due to local conditions.

5.3.3. To define seismicity of a region by the probability of a given level of spectral density of acceleration being reached in that region. The seismicity of the different regions must be established by combining geophysical, geological and seismic information and by using, as far as possible, quantitative statistical criteria.

5.3.4. To pursue analytical and experimental studies, based on the theory of random vibrations, mainly for obtaining further information concerning the behaviour of linear and non-linear systems with several degrees of freedom.

5.3.5. To complement the installation of strong-motion accelerographs for recording soil accelerations, by installing equipment also allowing to observe the behaviour of the structures themselves. Useful information may also be obtained by dynamic tests of real structures, even under vibrations of small amplitude.

5.3.6. The improvement of building codes is most desirable. As for wind, it would be convenient that codes contain simplified rules for the design of ordinary structures and basic data to be used in special studies of important structures.

5.4. Discussion is also open on dynamic loads other than wind and earthquakes. Among these, loads due to traffic, machinery and blast were mentioned. New data established on modern scientific bases shall largely contribute to the design of more economical and safer structures.

5.5. Finally, the importance of international collaboration as a powerful means of accelerating progress must be emphasized. This collaboration may be particularly fruitful for the establishment of recommendations of general character on which regional codes may be based.

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