DESIGN OF EARTHQUAKE - PROOF BUILDING STRUCTURES IN THE USSR

By

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INTRODUCTION

While establishing seismic regions on the territory of the USSR, it was found out that in the country there are about 5,5 sq.km. of land where earthquakes whose intensity is equal to 6 - 9 Mercalli-Sieberg seismic scale. Some 50 million people inhabit this territory. Naturally, the huge size of the capital construction according to the 7-year plan of the development of the USSR national ecomony makes exceedingly important the problems of prviding stability of buildings and structures during possible earthquakes. For eleven Union Republics (of fifteen Union Republics forming the USSR) problems of antiseismicity are problems of state importance. These problems are regulated by special All-Union Standards and Codes for building, having obligatory force for seismic regions on the territory of these Union Republics. The total sum of 3 trillion allocated for building according to the seven-year plan of 1951 - 1965 is distributed so that 25 percent of the capital investments are spared for building in seismic regions of the Union Republics. Therefore building economy is one of significant factors in seismic regions. The main problem is to lower additional costs for antiseismic measures, caused by the necessity of life protection during earthquakes and preservation of their labour results by means of building antiseismic structures capable to with stand earthquakes without destruction. Investigations of antiseismicity of buildings and structures in the USSR are carried on in 7 scientific research institutes located in seismic regions (Georgia, Amrmenia, Uzbekistan, Turkmenia, Azerbaidjan, Kazakhstan and Tadjikistan). Theoretical investigations of antiseismicity are carried on in Central Scientific Research Institute of Building Structures (Building and Architecture Academy of the USSR). This paper presents a survey of investigations carried on at the institute.

STRONG MOTION EARTHQUAKES FOR THE PERIOD OF 1902 - 1959

14 strong motion earthquakes were registered on the territory of the Soviet Union in the 20th century; some of them caused death of people and destroying a number of buildings (see the Table).

Nos.	The Name of An Earthquake	Date	Note
1	2	3	4
1	Shemakhinskoye	1902	Many buildings were destroyed and damaged

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2	Andijanskoye	1902	Many buildings were destroyed and damaged. There were victims
3	Alma-Atinskoye (Vernenskoye)	1911	"
4	Leninakanskoye	1926	"
5	Crimean	1927	Many building were destroyed and damaged
6	Zangezurskoye	1931	"
7	Karpatskoye	1940	"
8	Chatkalskoye	1946	"
9	Ashkhabadskoye	1948	A great many buildings were destroyed, many people perished
10	Khaitskoye	1949	"
11	Mondinskoye	1951	Some buildings were damaged.
12	Kurilskoye	1952	Tsunami. Many buildings were washed away by waves. There were victims
13	Muiskoye	1959	Some buildings were damaged
14	Kamchatskoye	1959	Many buildings were damaged. Some buildings were destroyed. There were victims.

Thus, for the past period 1 strong motion earthquake occured every 4-5 years in the USSR. It can easily be compared with well-known periodicity of strong motion earthquakes in California (USA) and Japan.

State Standartization of Building in Seismic Regions of the USSR

Up to 1940 building in seismic regions of the USSR was regulated in some areas (Kazakhstan, Crimea and Transcaucasus) by Specifications published by local authorities which had not All-Union recognition. In 1940 for the first time was adopted "Instruction for designing of Civil and Industrial Buildings and Constructions erected in Seismic regions," which was used all over the USSR.

In 1943 it was somewhat revised for the conditions of war time and was used in this edition up to 1948, when it was replaced by more detailed "Specifications for designing of buildings and structures in seismic regions" (TY-58-48). In 1951 were published "Regulations concerning building in seismic regions" ПСП-101-51),[2] which were the first All-Union Normative Document obligatory for use in building in all seismic regions of the USSR. Finally, in 1957 were adopted new "Standards and Codes for Building in Seismic Regions" (CH-8-57)[1] which essentially differ from all former normative documents.

In this normative document there are nine sections and the appendixes. In the first section are given instructions concerning the seismicity of the region and design seismicity of buildings and structures. In the second section are given instructions concerning the planning of towns and villages. In the third section are given the most important new instructions concerning seismic loads for dwelling, civil, industrial and agricultural buildings and structures. In the fourth section are given antiseismic measures for industrial and civil buildings and structures, the majority of instructions being concerned with the provision of antiseismicity of stone buildings. In the fifth section are given instructions concerning the building of water-and sewer pipes. In the sixth and seventh sections are given instructions for the design and antiseismic measures for road and hydrotechnical structures. In the eighth section are given instructions concerning the building in rural areas. In the nineth section are given requirements concerning carrying out works and control for carrying out antiseismic measures.

In the appendixes are given maps of seismic regions and enumerated some places, seismicity being pointed out for them. We are going to state here not all the contents of the standard CH-8-58 but only considerations and grounds which refer to a new dynamic method of determining design seismic loads for buildings and structures. This method essentially differs from the so-called "static" method of design which is used up to the present time in many countries including Japan where it was worked out and proposed by F. Omori and K. Sano, well-known Japanese scientists, in the beginning of the present centure for practical use.

Practice introduced amendments into this method of design (which was used in the USSR up to 1957) by bringing in the correction factor $\mathcal{L}$[2] The design value of seismic force was determined by means of a formula

$$S = \mathcal{L} K_c Q$$

where $\mathcal{L}$ - a certain correction factor;

K_c - seismic coefficient;

Q - the weight of an element of the construction located on a certain level of the building or structure.

Inadequacy of such approach to the design of structures for seismic forces is universally recognized. In the USSR the first suggestion concerning the transition from the "static" method of design under the action of a shifting base was made by K. S. Zavriev and A. G. Nazarov in 1931.(3) They proposed to proceed from the premise that the earth motion during an earthquake may be liken to harmonic forced (beginning in accordance with the cosine law) oscillations. The discussion on this problem which began then did not lead to a practical result and the method of design remained as before based on "static theory".

Beginning with 1948 the Central Scientific Research Institute of Building Structures (Building and Architecture Academy of the USSR), previously ZNIPS, commenced the work on the improvement of the design method of a seismic load.

The consideration of the elastic system oscillations with "n" number of degrees of freedom under the action of a horizontally shifting base (Fig. 1) shows(4) that forces appearing in such system are equivalent to forces S_j, which later we shall call seismic forces and which can be expressed in the following way:

$$S_K = \sum_1^n S_{Ki} = \sum_1^n K_c \beta_{it} \eta_{Ki} Q_K$$

where S_{Ki} - force at the point k with the deflection of the system in i - main direction;

K_c - coefficient, determining the relation between the acceleration of a shifting base and the acceleration of gravity (g), i.e. the value determining the seismic intensity of the region;

β_{it} - coefficient, determining the dynamic effect of the displacement base action to the system in consideration in each i - main direction at each currant moment of time t;

η_{Ki} - coefficient taking into account the influence of the deflection form of the system in a corresponding i - main direction, equal to

$$\eta_{Ki} = \frac{X_i(x_K) \sum_1^n Q_j X_i(x_j)}{\sum_1^n Q_j X_i^2(x_j)}$$

$X_i(x_k)$ and $X_i(x_j)$ - deflection of the system at its free oscillations in i - main direction at the point k and all points j, i.e. at points corresponding to the location of loads in adopted design scheme (Fig. 1) of a structure;

Q_k and Q_j - weights of the masses at the point k and all the points j of the system;

n - number of freedom degrees of the system, i.e. number of places where loads Q are located;

g - gravity acceleration;

The expression (2) shows that the value of seismic force S_k presents the sum of series of addends (S_{ki}); therefore it is natural that the design which takes into account all addends of the series is more perfect. However, at such summerizing it is necessary to remember that multiplicators β_{it} are values which change depending on time, therefore the determination of S_k as the sum of addends is very embarrassing and practically unattainable.

The examination of earthquake seismograms shows that usually every earthquake begins with oscillatory processes of high frequency, which are similar to the impulse action, and then passes to oscillations with lower and lower frequencies. The influences of such oscillatory processes upon systems with various frequency characteristics of free oscillations will be various as it is known. The dynamic effect of the impulse upon rigid systems with high frequency of self-oscillations will be essential and its action will be almost simultaneous with the moment of the impulse appearance. On the contrary, the maximum deflection in systems possessing low frequencies will correspond to the action of subsequent part of the seismogram and they will begin later, only after repeated oscillatory action of the base.

Taking this into account it should be noted that maximum values of separate addends of the series (2), i.e. forces S_{ki} reacting to oscillations of the system in accordance to its various forms of free oscillations usually possessing essentially different frequencies will also be mutually shifted by time. Seismic forces corresponding to a high frequency of the structure appear and get their maximum value at that moment when oscillations corresponding to a low frequency of the structure do not develop yet. To the moment when the latter get their maximum value the former will already be insignificant or completely damped (Fig. 2). Taking this into account it was decided that for practical purposes it was more expedient to consider separately all possible maximum values of forces in each main direction (Sji) and to design the structure for the most disadvantageous of all possible combinations.

In this case the necessity of determining the time variable β_{it} falls away and only its maximum value, which we shall write without index t i.e. presents some practical interest.

Thus, principle scheme of the design of the structure for seismic influences, or to be more precise, the scheme of design for determining seismic loads consists in calculating several groups of possible loads corresponding to the forms of free oscillations of the system and determined by the formula:

$$S_{kl} = K_c \beta_l \eta_{kl} Q_k \quad (3)$$

In the formula written above coefficients K_c and η_{kl} are quite definite and do not require special explanation. The coefficient β_l depends on the base motion law whose character during the earthquake is rather complicated and intricate and in every case is somewhat different.

For the determination of β value it would be rational to use the method proposed by M. Biot (9), (10), with the help of which dynamic effect of an earthquake is found out experimentally on models. However, this method required lengthy time for its realization. The use of the available in the literature data corresponding to seismic records made on the territory of America without their even if indirect critical evaluation was somewhat risky. Therefore Soviet scientists not only carried out experimental investigations on the elucidation of the dynamic effect of earthquake on various structure models (investigations performed under the leadership of A. Nazarov in the Armenian SSR (6)) but analysed a number of seismograms.

The analysis showed that for the mathematic interpretation of soil displacement during an earthquake one may adopt the law of summary action of damped harmonic curve series, i.e.

$$y_0(t) = \sum_1^n a_0 e^{-\varepsilon_0 t} \sin(\omega t + \gamma) \quad (4)$$

where $y_0(t)$ - soil displacement

a_0 - initial amplitude of the corresponding addend of the series.

ε_0 - coefficient of soil damping;

ω - frequency of oscillations of the corresponding addends of the series;

γ - the angle, determining the moment of appearing the corresponding addend of the series

t - time.

As is known the summary action of such harmonic curves with different parameters (i.e. different initial amplitudes a_0, frequencies ω damping ε_0 appearing at various moments of time) may cause diverse changes in the soil displacement character, practically defining any law of motion. The adoption of such soil displacement law connected with sudden change of velocity speed from zero to some final value ($a_0 \omega$) at the moment of appearing of the corresponding addend of the series (4) cannot be considered quite probable, though it quite well agrees with the conception of impulse sourse of generating oscillations and agrees satisfactorily with experience.

The possibility of practical usage of adopted soil displacement law for designing structures for seismic influence required the presence of the data of parameter values such as the components of equation (4). Processing a series of seismograms registered by seismic stations in various points of the USSR* showed that parameters characterizing equation (4) are placed within rather close value limits. The periods of oscillations of the earth surface (i.e. $T = \frac{2\pi}{\omega}$) corresponding to maximum accelerations were equal to 0.25 - 0.75 sec. and mean values of decrements of damping ≈ 0.10.

Thus as an initial premise for determining β - coefficient it is supposed that the motion of soil is submitted to the law of equation (4) in which the values $\frac{2\pi\varepsilon_0}{\omega}$ (the decrement of damping) in all addends are constant while ω - values may be of any magnitude within certain limits $\omega_1 - \omega_2$. Proceeding from these premises and making some assumptions about possible properties of series (4) and considering that the intermediate meaning of the value characterizing the damping of oscillations in building constructions (decrement of damping) is approximately equal to 0.30 there were determined β - values for systems with various periods of free oscillations T. The results of calculations are presented in Fig. 3a.

While discussing the project of standards for design of structures for seismic action which are used at the present time (1), into β - diagram (Fig. 3a) some corrections were introduced and finally it took the form presented by solid line (Fig. 3b)

It is interesting to compare β - diagram obrained with the diagram of seismic coefficient c which was adopted at California Project of Standards (8) and which was based on rich experimental material of many American earthquakes. However, it is necessary to make a reservation that c - diagram mentioned above presents some value somewhat different from β - coefficient.

C - value is the relation between maximum acceleration occuring in a structure with the period of natural oscillations T and the acceleration of gravity. Consequently, c - value is to the product of coefficients $K_c\beta$ but not to β.

In Fig. 36 there are given c - values and products $K_c\beta$ corresponding to $K_c = 1/40$. It is easy to note that values c and $K_c\beta$ although they are obtained in different ways are highly similar. Apparently such result of comparison should be considered favourable, for coincidence of results observed is the testimony of probability of approaching the value of dynamic action of earthquakes on structures to its real effect.

The analysis of design of real structures upon seismic action shows that for a great majority of buildings only the lowest mode of oscillation is of practical importance and consequently the design of such structures may be limited by the determination of only one group of forces S_{kl}.

* The seismograms which were used fitted for very small earthquakes; we could use only these seismograms because when shock were stronger the seismograms available became unfit for processing in view of large magnification and low speed of the tape motion.

The highest modes of oscillations, the second and in some cases the third, are essential mainly for flexible structures where belong chimneys, towers, masts, etc. Therefore the structures of this type should be designed for two or three combinations of seismic forces. Besides, β - value for the structure of this type is adopted 1.6 times greater taking into consideration the fact that they as a rule possess lesser damping.

A number of buildings subjected to the action of the earthquake the intensity of which was equal to 9 (seismic scale) after careful examination was subjected to control design according to the method described above. 14 brick buildings the height of which was equal to 1-4 stories, a reinforced concrete three-storied frame, a mill-structure (of compound construction), steel tower, supporting a water-tank, and a brick chimney 60 m high were subjected to such design. The results of control designs showed that in most cases ($\sim 85 - 90\%$) actual data concerning the destruction and safety of structures after an earthquake coincided with the results of the design. In particular it is necessary to point out the design of a brick chimney; in accordance with it the chimney should have destroyed in its upper part but not at its base where according to the former methods of design maximum stresses should be expected. In reality the upper part of the chimney destroyed.

Passing to the evaluation of the method described from the point of view of its practical fitness it should be pointed out that its main drawback is complexity and expenditure of much labour for calculations connected with the determination of periods and modes of free oscillations in various buildings and structures. If these two characteristics of a structure are known the further calculations are simple and do not require much labour. Therefore in the standard which is used at present and is called "Norms and Codes of building in seismic regions (CH - 8 - 57)" (1), together with basic principles of the method, there is a number of points which simplify to a considerable degree the technique of calculating in practical designs. For the determination of seismic forces in brick buildings which are used as dwellings there are given final values of products $\beta\,\eta$ and consequently there is no need intermediate calculations for determining periods (T) and modes($X_{(x)}$) of oscillations of these structures. Further instructions are given permitting the simplification of calculations connected with the determination of forms of structure deformations, etc.

For the interpretation of standards in detail an instruction is worked out at the present time, it contains formulae and diagrams which make easier the calculations of T and $X_{(x)}$ for various types of civil and industrial buildings.

In Fig. 4 and 5 there are given data of the design of reinforced concrete chimney, 80m high (5). In Fig. 4 the modes of the chimney oscilations (4a), the corresponding values of seismic forces (4b) and shears (4c) and bending moments (4d) are presented. As the design is to reveal maximum forces of envelope shears and moments are drawn in Fig. 5.

On the Leading Principles of Designing Antiseismic Buildings

The main principle of designing antiseismic buildings and structures is dynamic design for seismic foces the grounds of which are given above. Indeed, this method of design makes possible to obtain more plausible scheme of real behaviour of building structures during earthquakes. Leaning upon this relatively plausible design the USSR Standards (1) contain indications concerning some general principles of designing antiseismic buildings. While designing buildings for seismic regions it is necessary to follow the principle of equal strength. The analysis of many designs of antiseismic buildings shows that this principle is often neglected the result of it being weak joints and elements which may be the cause of the destruction of the building before the moment when the rest of the elements of the structure exhaust their safety factor. Therefore the standards recommend to obtain as similar strength as it is possible in all elements which form the bearing construction of the building. Effective distribution of masses and rigiditiesis the essence of the following principle ensuring antiseismicity of a building. The importance of this principle is great as its application permits to reduce the values of seismic foces to their minimum. It is important to take into consideration plastic deformations of constructions occuring under seismic action.

Experimental investigations for some time past confirm the results obtained at our Institute as far as 1953 (11), (12), showing that plastic deformations which are being developed give possibility to increase essentially the strength of materials at momentary impulse action which is known are a highly characteristic type of load in the very beginning of the earthquake. In certain frame structures deformations at impulse seismic influences may be rather large (13). While designing reinforced concrete frames it is necessary to create favourable possiblilities in them for forming plastic deformations during momentary action of an earthquake. Practically it means the striving to use the least possible percentage of reinforcement compensating it by increasing the size of the section, etc.

In consequence of the constant widening of the usage of built-up reinforced concrete structures in seismic regions the principle of making built-up reinforced concrete structures monolithic is the matter of prime importance. In other words it signifies the demand for turning a built-up structure into a structure equivalent to monolithic reinforced concrete (14), (15). In a special report these problems are examined in detail.

Some Data Based on Experimental Investigations

a) During an earthquake in structures a marked dynamic load occurs, its action is short at repeated loadings the number of which is approximately equal to 100 - 100 cycles. At present there are rather few direct experimental data which give the possibility to judge of the material strength under not numerous repeated loadings. Experimental investigations of the steel strength at not numerous loadings and speeds close to those .

ones which are observed in structures during earthquakes.

Specimens of round form, 10 mm diameter, 82 mm length, were tested on the pulsator of unilateral action. The purpose of the investigation was the establishment of the interrelation between the strength of the material and the number of cycles of loading. In the work of one of the authors (16) a suggestion was made that the interrelation between the value of the destructive stress and the logarithm of the number of repeated loadings is linear. The experiments made confirmed for steel specimens the existance of such dependance. In Fig. 6 experimental confirmation of the hypothesis adopted is illustrated (17); along X - axis the logarithm of the number of loading cycles and along Y - axis the relation between the stress at dynamic destruction of the specimen and its limit stress at static loading were marked. Hence it is seen that at short action of a few repeated loadings the steel strength increases by 20 percent. This result is taken into consideration in Standards (1) by introducing an additional coefficient raising stresses in steel during an earthquake.

b) According to the new method for the determination of design seismic load on the building the calculation of periods and modes of free oscillations is required (1). While determining theoretically of these parameters for various buildings one meets certain difficulties (8). In order to derive formulae, well-grounded and simple, practical and theoretical investigations of free oscillations of buildings with load-bearing stone walls were carried out. Tests of such buildings were carried out with electro-dynamic vibrograph and the records were made with the oscillograph (18; 19). In Fig. 7 the records of oscillations of a 6-story building with the variable thickness along the height of load-bearing walls. One of the records presents the second form of oscillations. Theoretically this problem was solved with the consideration of elastic clamped building in the ground. As a result we obtained the formula for the determination of free oscillation periods for such type of a building:

$$T = \frac{9.75\,H}{\alpha_i}\sqrt{\frac{m}{FG\gamma_0}}$$

where H is the height of the building in cm; α_i - is the frequency coefficient which is determined from the diagram in Fig. 8 and which depends on the interrelation of rigidities of the building and the ground; i - is the mode of oscillations; m - is the mass in kg. sec.^{2}/cm; F - is the area brutto of walls in the plan in cm^2; G - is modulus of elasticity of masonry for shear in kg/cm^2 which at low oscillations is equal to 0.4 E and at high oscillations is equal to 0.25 E where E is modulus of elasticity of masonry for bending; γ_0 - is the coefficient taking into account the area of openings in walls. We obtained rather simple formulae which agree satisfactorily with a series of field measurements of periods:

$T = 0.0165$ H - brick buildings

$T = 0.014$ H - big block buildings

where H - is the height of the building in m.

In Fig. 9 there are given comparisons of measured (Fig. 7) and calculated periods of oscillations of a 6-story building. It is necessary to point out that the difference for the first two modes is equal to 3 percent and 1 percent, respectively.

c) The development of experimental investigations of small models of buildings and structures tested on seismic platforms is to our mind rather an effective means for solving a number of problems concerning designing and building of antiseismic structures.

This paper presnets a brief summary of experimental investigations carried out on small three-dimensional models of interconnections of brick walls of buildings, some types of hydrotechnical structures made of large concrete blocks and earth dam of slope type.

The investigations carried out are some experience of solving certain problems concerning antiseismicity of building and structures by means of experimental investigation of their three-dimensional models.

More than 200 models of various types were tested on a seismic platform with photoelectric program control.

Experiments with models of the interconnection of brick walls (20) were carried out by comparing the behaviour characteristics of two models which were under the action of horizontal oscillations at their base. Both models were equal in their external sizes and materials but differed constructively. The experiments showed that strengthening the construction of walls only in horizontal joints does not provide their stability during horizontal oscillations. The stability and resistivity of wall interconnections increases greatly with vertical reinforcement of the construction (Fig. 10).

The measurements of horizontal oscillation amplitudes on the levels of floors (Fig. 11-12) in the stage of destroing models of interconnections of walls 1 - 4 stories high (21) gave amplitudes on the level of floors which are shown in Fig. 12.

Experiments with two similar models of wall intercommections which are situated on pliable and ridge ground showed that at horizontal oscillations in the base with the ranges of frequency equal to 1.40 - 4.75 c/sec. both models vibrated and the frequency of oscillations was rather close to the frequency of their free oscillations.

The frequency of free oscillations of the model on ridge ground exceeded the frequency of free oscillations of the model on a pliable ground by 42 percent. The pliability of the ground appears to lead to some decrease of the structure free oscillation frequency (22).

Experiments of models of slope-type big massive-fill hydrotechnical structures showed their considerable stability during the action of horizontal oscillations in their base as compared to the models of similar concrete block masonry structures having longitudinal vertical joints. In these last models considerable damages and destruction caused by the lamination of masonry along vertical joints were observed. Walls of concrete block masonry where concrete blocks span the whole wall width without vertical joints showed the best stability (20).

Experiments with models of construction of embankment walls built of usual concrete block and having vertical joints showed that they do not possess required stability when they subjected to strong horizontal oscillations in the base. The principal cause of their destruction consisted in the lamination of masonry and considerable displacement of concrete blocks leading to their destroying (20).

Models of sheet-piling type of structure with anchor devices and some other types of wall constructions showed the best stability (20).

Experiments with models of an earth dam (20) showed that the principle condition providing the seismic stability of the dam at acceleration range of 0.03 - 0.30 g is the initial porosity which is approximately equal to 36-37 percent for fine and small grain sands which were the building material for the dam model. At such initial porosity slopes 1 : 2 appear to be stable (Fig. 13-16).

As a result of these investigations certain regularities in horizontal oscillations in models of interconnections of brick walls and walls made of regular concrete blocks were elucidated, which ascertain the character of displacements according to the height of the model. Characteristic distinctions of the behaviour of concrete block masonry in aerial and water medium and regularities in changing setting, porosity and slopes in a dam built of sand (of not connected earth materials) were established with their dependance upon time.

The Perspectives of Developing Theories of Antiseismicity and Antiseismic Engineering

As a result of work of a number of Soviet and foreign scientists (J. L. Korchynski, A. G. Nazarov, B. K. Karapetjan, M. G. Ourazbajev, S. V. Medvedev and others (23), M. Biot, G. Housner, K. Kanai and others (9, 10, 26, 27, 28) in the USSR, the USA and Japan the dynamic theory the principle theses of which are stated above was developed to a great extent. Such development was caused by the establishment of some statistic data concerning the movement of soil during an earthquake and consequent reaction of various types of structures, this reaction being characterized by dynamic parametres of structures. It if proved that at an average the value of acceleration which is passed to the pendulum by an earthquake depends upon the period of free oscillation of this pendulum. There is a general statistic regularity according to which maximum accleration of the pendulum noticeably decreases with the increase of its period. The essence of a new dynamic theory of antiseismicity consists in direct usage (while working out methods of structure design) of this new characteristic of the earthquake process. The new method of design is undoubtedly nearer to reality than all the previous methods and its introducing into Normatives (1) is progressive. However, this method insufficiently and not quite consistantly uses the statistic character of the process (of an earthquake and response of the structure) which is dealt with in the present case.

It stands to reason that at the present time when for characterizing earthquake processes we have only one spectral curve there is no possibility to propose anything better than this method. However, as new statistic data concerning an earthquake process are accumulated a new more perfect theory may be developed (24). Modern science possesses methods which give the possibilities to reveal regularities typical of similar stochastic (non-stationary) processes, therefore the problem is to spread these methods over studying the process of the structure deformations during earthquakes. The purpose of the stochastic theory of antiseismicity is the determination (according to statistic characteristics of an earthquake process) of statistic characteristics of deforming structure elements and the indication of reasonable limitations for them. Further development of antiseismic theory seems to follow this way (25). However, a new dynamic method of design given in the Norms (1) should be perfected and assuming it as basis it is necessary to work out new constructive solutions satisfying seismic conditions to the utmost.

Structures with the minimum possible weight the decrease of which must be carried out at the expense of light roof and floor constructions and diminishing lateral rigidity in the wall plane are the most perspective. The use of plastic and aluminium roof constructions for frame buildings of industrial shops leads to loads approximately equal to 20-25 kg/m^2 instead of 350 - 500 kg/m^2. In such case considerable decrease of seismic force values leads to rather effective decrease of the cost of building, as in a number of cases a wind load may be prevalent in the combination of loads. This means that additional antiseismic measures can be reduced in cost to quite moderate expenditures (29). Still greater economical effect can be achieved for frame buildings taking into account plastic deformations of constructions. As a preliminary analysis shows (13) plastic deformations may be considerable without destroying the frame construction. Undoubtedly working out a method of construction design for seismic action with the consideration of plastic deformations is one of most urgent problem of anti-seismicity. The idea of designing such antiseismic structures in which it is possible during an earthquake to allow large deformations (foreseen by the design and constructing) and even destruction of separate secondary elements if these deformations and destruction do not cause demolition the whole construction must appear to be perspective.

REFERENCES

1. "Standard and Codes for Building in Seismic Regions (CH-8-57)," Gosstroyisdat, 1957.

2. "Regulations concerning building in seismic regions" (ПСП-101-51), Gosstroyisdat, 1951.

3. Zavriev K.S., To the Problem of Antiseismic Theory. Tiflis, 1933.

4. Korchynski J.L., The Design of Structures for Seismic Loads, Gosstroyisdat, 1954.

5. Korchynski K.L., Seismic Loads on Buildings and Structures, Building Industry Scientific and Engineering Socity, Moscow, 1959.

6. Korchynski J.L., Soudnitsin A.S., and Bikhovsky V.A., The Design of Reinforced Concrete Chimneys Built in Seismic Regions. Journal "Concrete and Reinforced Concrete," No. 10, 1957, Moscow.

7. Nazarov A.G., Method of Engineering Analysis of Seismic Forces. Academy of Science, Armenian SSR, Erevan, 1956.

8. Tsshokher V.O. and Bikhovsky V.A. Antiseismic Building, 1937, Moscow.

9. Lateral Forces of Earthquake and Wind. By a Joint Committee of the San Francisco, California Section, ASCE, and the Structural Engineers Association of Northern California. (Proceedings American Society of Civil Engineers, April, 1951, Vol. 77, Separate No. 66).

10. Biot, M.A. A Mechanical Analyzer for the Prediction of Earthquake Stresses, Bull. of the Seism. Soc. of America, Vol. 31, 2, April, 1941.

11. Biot, M.A. Analytical and Experimental Methods in Engineering Seismology, Proc. American Society of Civil Engineers, Vol. 108, 1943.

12. Goldenblat J.I. and Bikhovsky V.A., Actual Problems of Present Day Antiseismic Building. Building in Seismic Regions. Building Industry Scientific and Engineering Society, Moscow, 1956.

13. Gvozdev A.A., To the problem of design of structures for the action of explosive wave. Journal Building Industry, Nos. 1-2, 1943, Moscow.

14. Nikolaenko N. A., Plastic deformations in problem of dynamic design of structures. Investigations on antiseismicity of buildings and structures. Building and Architecture Academy of the USSR, ZNIISK, Gosstroyisdat, 1960.

15. Churayan A.L. and Dgiabua Sh.A., Some problems of using built-up reinforced concrete in antiseismic building. Academy of Science, Grusinskaya SSR, Tbilisi, 1956.

16. Bikhovsky V.A., The Use of Built-up Structures in Seismic

Regions, Academy of Science of USSR and Building and Architecture Academy of USSR, ZNIISK, 1958, Moscow.

17. Korchynski J.L., The bearing capacity of materials at not numberous repeated loadings. Bulletin of Building Technic, No. 3, 1958, Moscow.

18. Korchynski J.L. and Becheneva G. V., The Strength of Steel at not numerous repeated loadings. Journal of Building Mechanic and Design of Structures, Building and Architecture Academy USSR, No. 1, 1960.

19. Korchynski J. L. and Pavlick V. S., Field Investigations of Building Vibration Proceedings of Conference, Leningrad, 1958.

20. Pavlick V. S., The Determination of Free Oscillations of Buildings with Bearing Walls. Investigations on antiseismicity of Buildings and Structures, Building and Architecture Academy of USSR, ZNIISK, Gosstroyisdat, 1960, Moscow.

21. Bikhovsky V. A. and all., Investigation on Anti-seismicity of Buildings and Structures, ZNIPS, Gosstroyisdat, 1956, Moscow.

22. Bikhovsky V.A. The Degree of Similarity of Deformations in Three-Dimensional Models of Wall Interconnections in Brick Buildings, Dynamic Investigations, Building and Architecture Academy of USSR, ZNIISK, Gosstroyisdat, 1956, Moscow.

23. Bikhovsky V. A., Experimental Investigations on Dynamic Behaviour of the Models on Pliable and Ridgid Ground. Investigations on Antiseismicity of Buildings and Structures, Building and Architecture Academy of USSR, ZNIISK, Gosstroyisdat, 1960, Moscow.

24. Methods of Design of Buildings and Structures for Antiseismicity. Building and Architecture Academy of USSR, ZNIISK, Gosstroyisdat, 1958, Moscow.

25. Goldenblat J. I., On the Possibility of Developing Stochastic Theory of Antiseismicity. Ibidem.

26. Goldenblatt J. I. and Bikhovsky V. A., On the Development of Methods of Design of Structures for Antiseismicity. Ibidem

27. Housner, G. W., Spectrum Analysis of Strong Motion Earthquakes.

Proceedings of Symposium on Earthquake and Blast Effects on Structures, Earthquake Engineering Research Institute, Los Angeles, 1952.

28. Housner, G. W., Martel, R. R., and Alford, J. L., Spectrum Intensities of Strong Motion Earthquakes. Bulletin of the Seismological Society of America, Vol. 43, 1953.

29. Kanai K., A Study of Strong Earthquake Motions, Bulletin of the Earthquake Research Institute, University of Tokyo, Vol. 36, Part 3, IX 1958.

30. Nassonov V. N., Bikhovsky V. A., Giabua Sh. A. Dusinkevitch S. Ju., Korchynski I. L., Polyakov S. V. and Stepanian V. A., Ways of decreasing Cost of Industrial Buildings Erected in Seismic Regions, Journal of Building Industry, No. 8, 1959, Moscow.

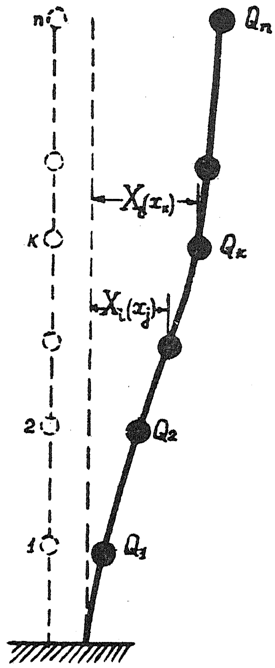


Fig. 1. Design scheme of elastic system with "h" degrees of freedom which is acted upon by a horizontally displacing foundation.

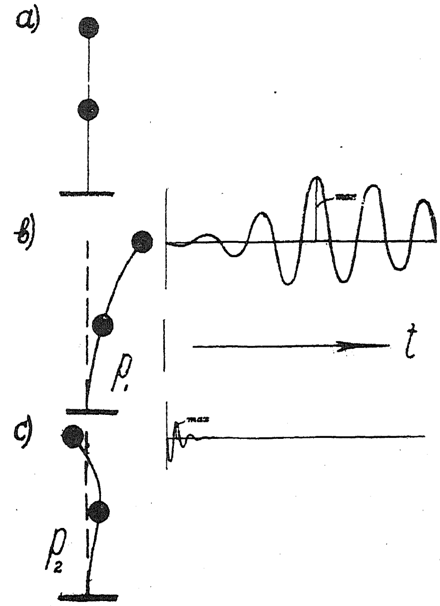
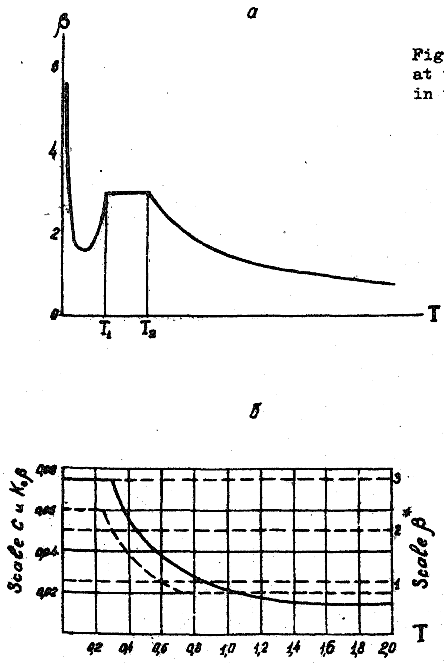


Fig. 2. Deflections of the system with two degrees of freedom at two first modes of free oscillations generated by movement in the foundation of the system.



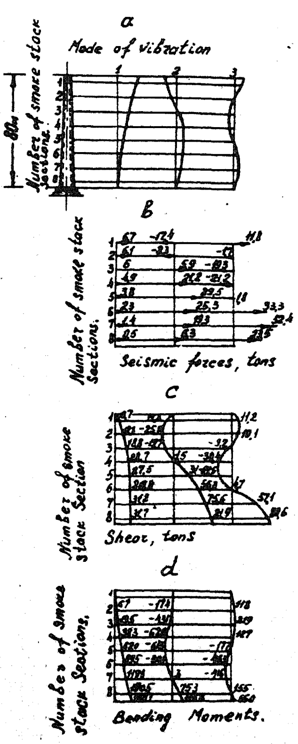


Fig. 4
The design data of seismic foces, shears and bending moments for a reinforced concrete chimney, 80 m high;
a - oscillation modes;
б - seismic forces;
c - shears;
d - bending moments

Fig. 3
a) The diagram of of values β for systems with various periods of free oscillations.

b) The comparison of diagrams of β and C coefficients.

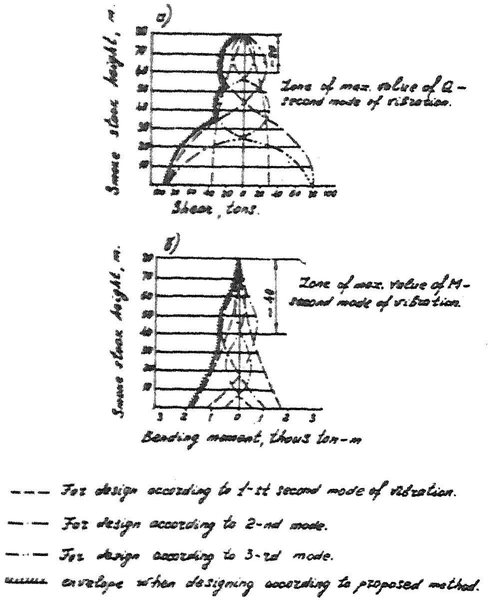


Fig. 5. Envelope epures of forces;
a - shear;
б - bending moment.

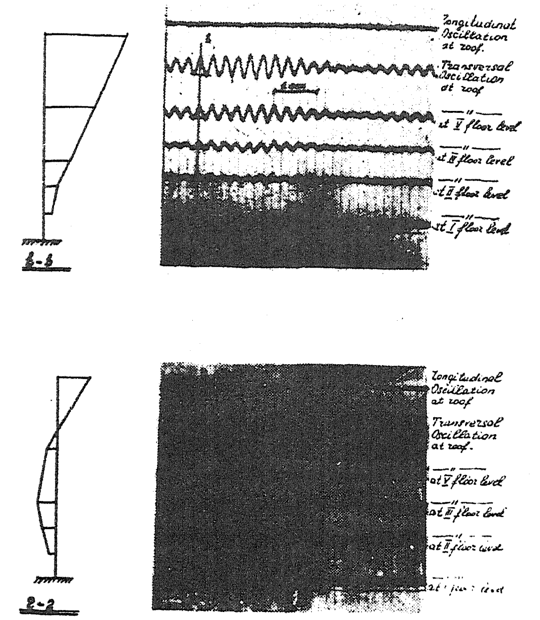


Fig. 7 Records of oscillations of a six-storied buildings and the corresponding modes of oscillations: the first (b-b) and the second (2-2).

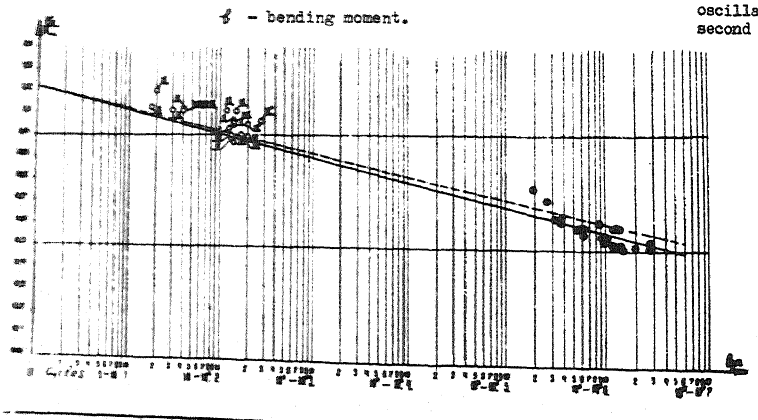


Fig. 6
Lineal relation between the value of destroying /limit/ dynamic stress and the number of loading cycles. The solid line - theoretical relation at asymmetry coefficient equal to zero. The dotted line - theoretical relation at asymmetry coefficient equal to 0.143.
• - experimental data on fatigue
o - experimental data at not numerous repeated loadings

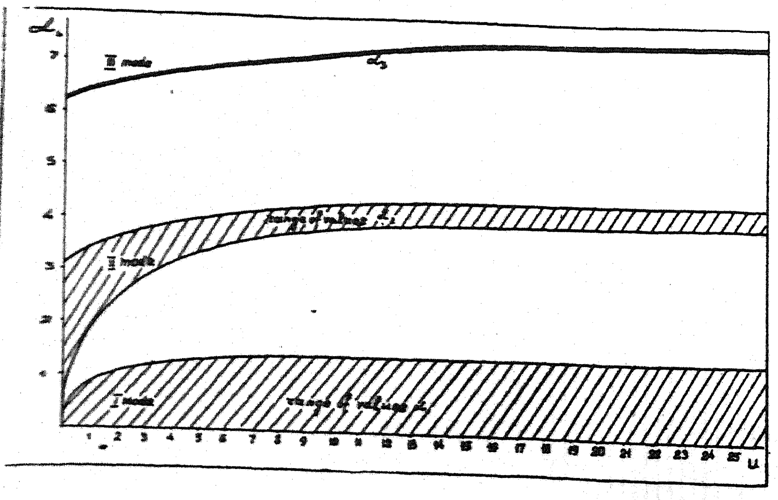


Fig. 8
The diagram: frequency coefficients (α_i) for three lower oscillation modes

$U = \frac{K_1 K_\psi}{FG} H \qquad V = \frac{K_1 K_\varphi}{HFG}$

$\frac{K_1}{FG}$ - is the regidity characteristic of the building;
K_φ - the rigidity of the foundation at turn;
K_ψ - the characteristic of the foundation rigidity at shear shear;
H - the height of the building.

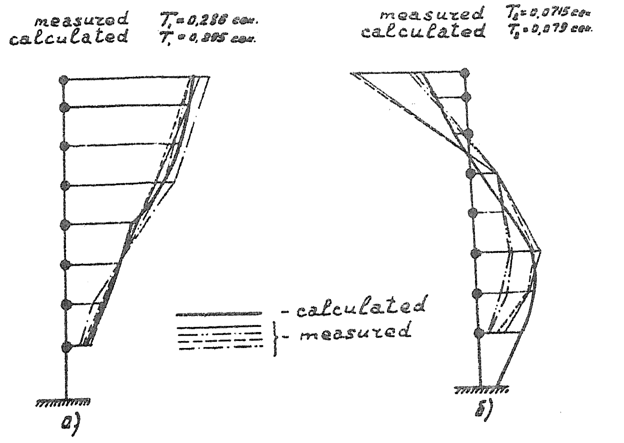


Fig. 9. The comparison of measured and calculated periods and modes of oscillations of a six-storied building with a varying width of walls.

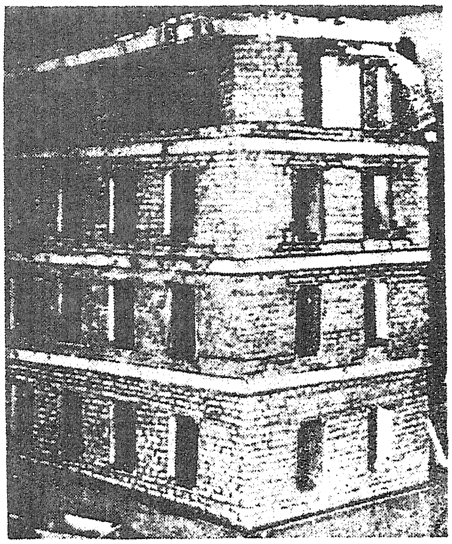


Fig. 11. The character of destruction of a four-storied brick building model.

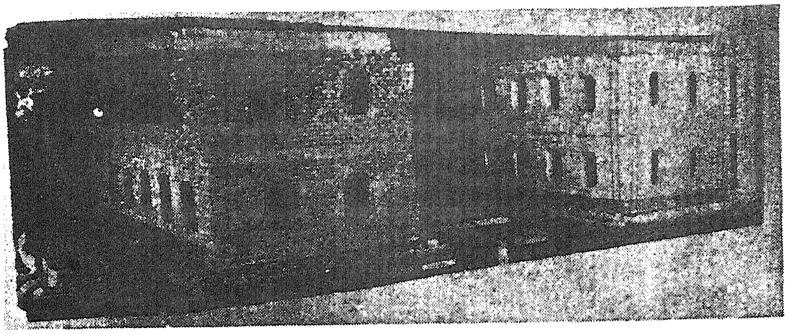


Fig. 10. Two simultaneously tested models of brick two-storied buildings, one model being reinforced vertically.

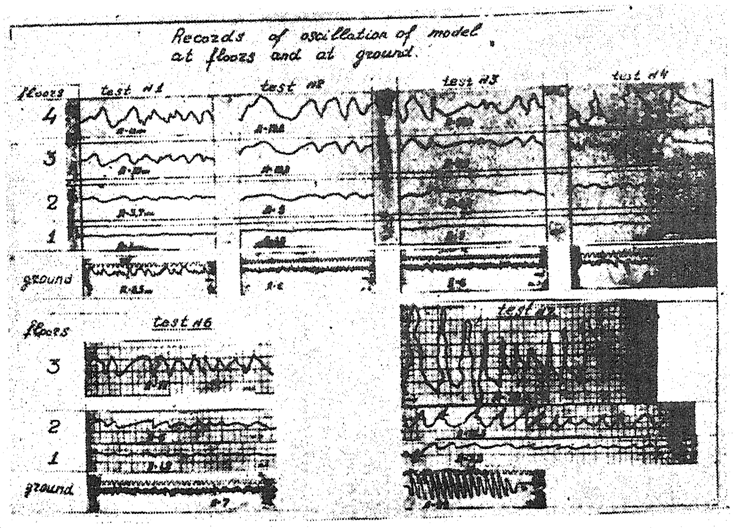


Fig. 12. Vibrograms of recorded oscillations of models on various stories and at the foundation.

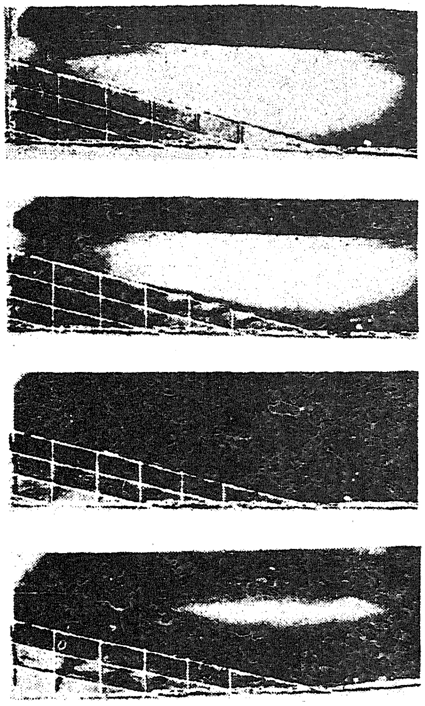


Fig. 13 The charater of deforming the slope 1:4.

Model N18

Initial data

Scale of Model – 1/100
Volume – " – 42840 cm³
Weight of dry sand – 62,118 kg
Moisture of sand – 23,18 %
Weight of moisture sand – 76,518 kg
Specific weight of model – 1,78 g/cm³
Porosity – 45 %

Test NN	Duration sec.	T sec.	A mm.	a/g
1	8,5	0,469	5,5	0,10
2–3	39,7	0,469	5,5	0,10
4	4,0	0,400	32	0,80

	after test NN		
	1	2–3	4
Volume of model, cm³	38994	37498	37498
Specific weight of model g/cm³	1,97	2,04	2,04
Porosity %	39,8	37,4	37,4

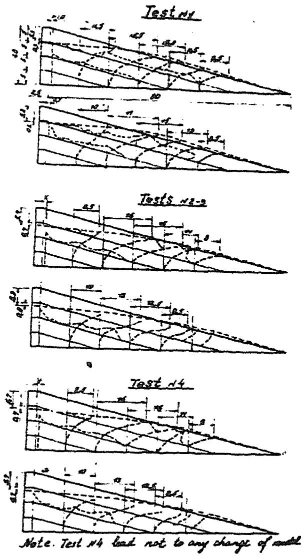


Fig. 14 The same. The dotted line shows the situation of coloured parts of sand after the corresponding oscillations.

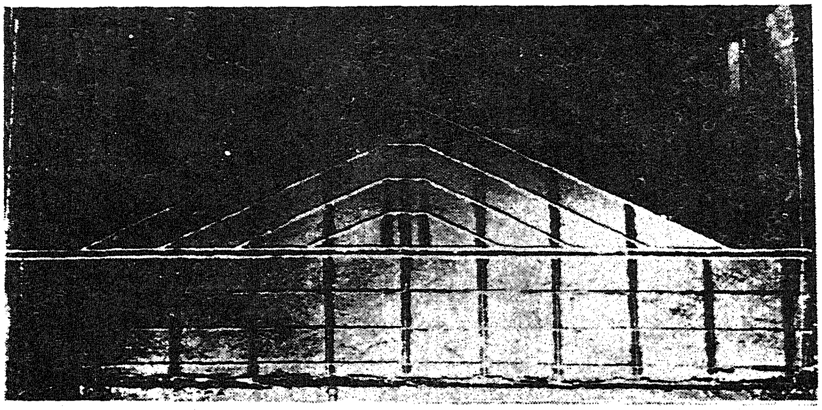


Fig. 15. The model of a dam with slopes 1:2 on soft base at porosity equal to 45 percent.

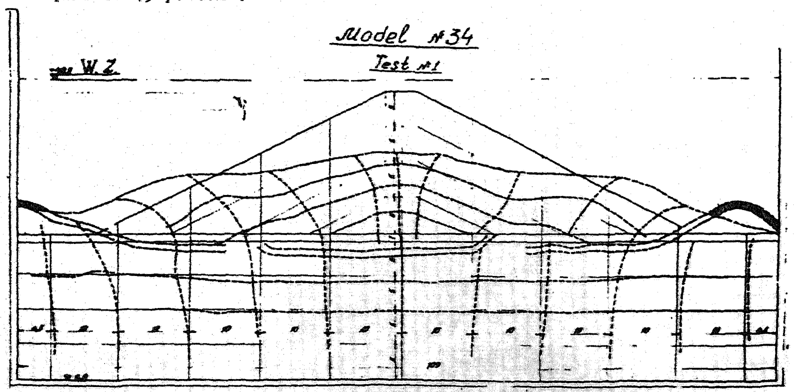


Fig. 16. The character of deforming the model shown in Fig. 15. The dotted line indicates the situation of coloured parts of sand.