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On the Seismological Basis for the Seismic Building Code in Russia

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Abstract—Based on the current ideas of engineering seismology, the paper discusses several problems involving seismology and earthquake-proof design that Russia faces at present. Many of these problems arise because the building code uses a macroseismic intensity scale as a characteristic of ground motions. The main conclusions are as follows. The macroseismic intensity scale is suitable for general seismic regionalization (GSR). The conventional formula relating the intensity to ground motion amplitude should be revised. A GSR map should reflect the intensity of a bedrock, rather than medium, ground. The description of the effect of ground conditions in terms of intensity increments is obsolete and should be replaced by the spectral description. The use of intensity and incremental intensity values rounded off to the nearest integer must be rejected.

INTRODUCTION

Russia is one of a few countries that entered the 21st century with a general seismic regionalization (GSR) map and building norms expressed in terms of macroseismic intensity. Moreover, except for special facilities such as nuclear power stations, the cooperation between Russian seismologists and civil engineers is traditionally conducted in terms of medium-ground and incremental intensity. The question arises as to whether Russia, following the majority of other countries, should adopt the language of physical parameters describing ground motions (amplitudes, spectra, etc.), or the traditional approach possesses hidden advantages. This problem is discussed below.

The initial and immediate task of the macroseismic intensity scale is to serve as a means for a noninstrumental determination of relative values of ground motions produced by an earthquake. The scale itself by no means implies its use for *normalization of seismic loads* in earthquake-resistant civil engineering. It is true, however, that several macroseismic scales are correlated with specific ranges of ground motion parameters, but this is made solely for orientation, so that a concrete recorded parameter of ground motions does not determine the intensity value. However, a stable tradition of using macroseismic scales precisely for the normalization of the seismic load has stroke roots in Russia over the last half-century. In this context, a significant, although essentially secondary, problem of correlation of physical parameters of ground motion with intensity values becomes unexpectedly important.

The normalization of seismic loads in its present form can be briefly described as follows. A map of seismic hazard (*sensus stricto*) is constructed for the entire territory of country. Below, the combination "seismic hazard" is used as a term. The seismic hazard is closely associated with the notion of the seismic shaking capacity introduced by Riznichenko [1965] (see also [*Seismicheskaya* ..., 1979]. The shaking capacity is defined as the (year-averaged) recurrence of shocks B of an intensity I' equal to or exceeding a given threshold value I. The dependence of B on the threshold intensity I is a decreasing function $B(I)$ expressed in units of yr^{-1}. However, seismic hazard maps fix the values of the inverse function $I_{d0} = I(B_0)$ for a fixed value $B = B_0$ (e.g., for $B_0 = 0.002\ \mathrm{yr}^{-1} = 1/500$ yr). In other words, the seismic hazard map shows macroseismic intensity values I_{d0} for future earthquakes, the probability of their occurrence formally coinciding with the real value. The usually fixed probability level of shocks with intensities equal to or higher than I_{d0} means that at least one event occurs every 50 years. Thus, the current SNiP II-7-81* building code includes OSR (GSR) maps A, B, and C with respective probabilities of 10, 5, and 1% (for a 50-year period); they correspond to shaking capacities of B_0 = 1/475, 1/950, and 1/4750 yr^{-1} or to rounded recurrence periods of 500, 1000, and 5000 years [Strakhov *et al.*, 1998; Gusev *et al.*, 2000].

The seismic hazard maps mentioned above (primarily the OSR-97-A map for a 500-year period) are regarded as GSR maps of the country [Ulomov and Shumilina, 1999; *Seismicheskoe* ..., 2000] and are included in the building code of norms and rules [*SNiP II-7-81**, 2000]. A designer uses I_{d0} as a design intensity value (an input parameter for the structural design). If the GSR detail level is insufficient, detailed seismic regionalization (DSR) or its abridged variant of initial seismicity updating (ISU) are carried out. Such investigations are also aimed at obtaining design intensity values.

The mapped design intensity relates to a medium ground. At a concrete site, the design intensity is corrected for actual properties of ground:

$$I_d = I_{d0} + \Delta I,$$

where ΔI is the ground correction called an intensity increment, and the I_{d0} value is taken from a GSR map in accordance with the geographic position of the building site. The I_{d0} values are integers in a range of 6 to 9. The possible values of ΔI are –1, 0, or +1. In the simplest cases, ΔI is estimated from engineering-geology evidence on the site lithology. For this purpose, ground is classified into three categories: I (bedrock and other particularly dense grounds), II (medium grounds), and III (soft grounds). Based on the engineering geology alone, one has

$$\Delta I = (\text{ground category number}) - 2.$$

However, ΔI is preferably estimated the direct seismological method from amplitudes of motions of a given ground (relative to the medium ground).

The spectral response method is then used for converting I_d into a quasi-static design load. In the simplest case, a structure is modeled by an elastic single-mass with a dimensionless damping of 0.05. The following two steps are carried out within this design scheme.

(1) The design peak acceleration A of the ground motion under a structure is estimated by postulating a fixed functional relationship between A and the design intensity I_d. According to the current norms, A is a dimensionless value measured in the units of gravity acceleration g, and the above relationship is adopted in the form $A = 0.1 \times 2^{I_d - 7}$.

(2) The design inertial force applied to the structure is estimated as

$$S = KmRA(T) = KQA\beta_{\Delta I}(T),$$

where m, Q, and T are the mass, weight, and natural period of the structure; $RA(T) - gA\beta_{\Delta I}(T)$ is the design acceleration response spectrum (in essence, this is the effective inertial peak acceleration of the structure due to forced vibrations of the latter; for normalization purposes, it is represented as the product of two factors: the peak ground acceleration gA and the dynamic coefficient $\beta_{\Delta I}(T)$); $\beta_{\Delta I}(T)$ is a quantity empirically determined from the analysis and synthesis of processing data of real accelerograms and represents the averaged dimensionless ratio of the inertial peak acceleration of the elastic pendulum $RA(T)$ to the peak acceleration of the base of this pendulum gA; and K is a purely engineering correction factor (in the simplest case, it is 1.0 in the design of structures whose damage is unacceptable and 0.22–0.35 for ordinary buildings and structures).

Note that the force S has two meanings: by origin, it is a dynamic load produced by ground motions, but it is assumed static in the design; the very idea of the (simplified) engineering design reduces to the replacement of dynamic calculations by simpler static ones. The SNiP functions $\beta_{\Delta I}(T)$ are slightly different for the values $\Delta I = -1$, 0, and +1. The set of the functions $A(I_d)$ and $\beta_{\Delta I}(T)$ is the same throughout Russia's territory.

Response spectra are used in other countries as well; however, the relationship between intensity and design acceleration, as a rule, is not used, and the GSR map is constructed directly from design values of physical parameters of ground motions. Maps in the majority of countries show the design peak acceleration A, but the response spectra for two periods are directly mapped according to the most recent norms in the United States. As distinct from the MSK-64 intensity scale [Medvedev, 1968] and its improved variant MMSK-84 [Ershov and Shebalin, 1984] that are implicitly regarded as a Russian standard, the new European scale EMS-98 [Grünthal, 1998] basically contains no references to amplitude parameters of ground motions. It is widely acknowledged that no fixed relationship exists between the observed values of intensity and peak acceleration (as well as the observed real value of $RA(T)$). Thus, the principle of mapping intensity for the GSR purposes, presently adopted in Russia, in conjunction with the postulate on a fixed intensity–acceleration

An increase in the observed peak acceleration amplitude averages at $I = 9$ due to an increase in the number of recorded accelerograms

Year of studies	Accelerations		Velocities		Source
	a_{max} (range), g	a_{max}, g	v_{max} (range), cm/s	v_{max}, cm/s	
1942	–	0.32	–	–	[Gutenberg and Richter, 1961, p. 52]
1964	0.2–0.4	0.28	16–32	22	MSK-64 [Medvedev, 1968]
1973	0.24–0.48	0.34	24–48	34	Scale project of 1973 [The Scale..., 1975]
1975	–	0.52	–	–	Trifunac and Brady [1975]
1988	0.4–0.8	0.6*	50–100	70	Aptikaev and Shebalin [1988]
1999	0.65–1.24	0.92*	60–116	83	Wald *et al.* [1999]

* Extrapolated values.

relationship can by no means be regarded as self-evident or generally accepted and must be discussed.

Even more numerous problems arise in relation to the concept of the intensity increment. The idea of incorporating the ground effect in the form of a frequency-independent correction is itself acceptable as an engineering tool in some cases. The difficulty consists in the fact that, whereas the idea of the dependence of the design peak acceleration (for a medium ground) on the intensity in a GSR map (also for a medium ground) is relatively reasonable, the assumption on the direct dependence of the peak acceleration on the ground category (i.e., on the intensity increment) is basically invalid. In fact, for several reasons the observed peak acceleration is nearly independent of the ground type (see below). Therefore, the seismic load estimate based on the direct dependence of the peak acceleration on the ground category (at a fixed level of ground motions of a geological bedrock basement) is also incorrect. Below, I discuss the equally important fact that, given the large amplitudes of ground motions (an intensity of 8 and more on a bedrock ground), an increase in the damage to buildings on "bad" grounds is not necessarily accompanied by any increase in amplitudes.

The interest in the above problems is far from being purely speculative. An incorrect solution of a problem at the junction of seismology and earthquake-proof construction can lead to either dangerous errors in engineering design or expensive redundant reinforcement. The following key moments are discussed below.

(1) Can the traditional use of the intensity I as a basic parameter of normative GSR maps still be regarded as acceptable, or does an obviously better alternative exist (such as the peak acceleration, response spectrum, and so on)?

(2) If such a use is acceptable, then (a) is the concrete relationship $A(I_d)$ adopted in the current regulations adequate, and (b) should a GSR map show the intensity specified for the medium ground?

(3) Is the traditional description of the ground effect adequate in terms of intensity increments?

(4) Is the traditional roundoff (to an integer) of both the design intensity I_{d0} and its increments ΔI adequate?

INTENSITY AS A GSR TOOL: ADVANTAGES AND DISADVANTAGES

A distinct tendency in the GSR maps of the last decades is the replacement of intensity values by amplitude parameters of ground motions. Most countries use the peak acceleration of ground motions, and the United States adopted the response spectra. (The GSR in the United States is implemented as two RA maps at periods of 1 and 0.2 s, and the RA values at other periods are found by interpolation.) Is Russia behind the times? Can it be that the Soviet/Russian practice of showing the intensity in GSR maps is obsolete?

I describe the order of the GSR map application. An engineer converts the design intensity I_d into the design response spectrum $RA(T)$ in accordance with the rules elaborated by civil engineers, and this conversion supposedly reflects the relationship between the observed (with a given recurrence) real intensity and parameters of the observed ground motions. The following two problems arise here: (1) the observed macroseismic intensity I is not rigidly connected with the observed peak acceleration (the scatter in a_{max} at a fixed intensity is large and, moreover, the average dependence $a_{max}(I)$ can be determined with a large uncertainty); and (2) the observed a_{max} value cannot be identified with the design value of A. Both these problems need be discussed.

Thus, the scatter in a_{max} measurements at a fixed intensity is large, the standard deviation of $\log a_{max}$ being on the order of 0.3 (which is accurate within a factor of 2). However, in using the dependence $A(I_d)$ from the building code, the A-based value of a_{max} predicted for a future earthquake is rigidly determined from the I value. Is the error of such a prediction important from the engineering standpoint? Apparently, it is not. The only important thing is that the design value of A must be realistic and must monotonically and systematically increase with I_d. An engineer is not much interested in the true value of a_{max}; what he really needs is an adequate design basis of A ensuring the admissible strength of a structure. The relationship between the actual value of a_{max} with the A value used in the design is not trivial. First, the structure is not perfectly brittle, and a single acceleration pulse is not dangerous; more important is the amplitude level in the range of maximum amplitudes of an accelerogram (of the type of an rms acceleration extremum estimated in a moving window of a few seconds in width). Second, a very short impulse (e.g., with a characteristic frequency of 15 Hz) is also believed to be nondangerous, because important are values of a_{max} in the engineering range of periods (i.e., after applying a band-pass 0.3–10-Hz filter to the accelerogram). Third, the damage extent depends on not only the acceleration amplitude but also the duration of ground motions with amplitudes close to maximum values (because the fracture of a real structure occurs not instantaneously but gradually, by accumulating defects, which requires time). Due to all these factors, engineers often prefer to use an effective, rather than seismologically observed, peak acceleration, but a general approach to the determination of this effective parameter is still absent. In an ideal case, precisely this effective value should be used as a design basis of the peak acceleration A.

In this situation, it is reasonable to directly relate the A value (the only design parameter closely reflecting the load scale) to the intensity that primarily reflects the extent of damage to hypothetical buildings with a fixed vulnerability (e.g., such as the once-typical one-story brick house after Medvedev [1962]). Thus, it is clear that the use of the direct relationship between the design intensity I_d with the design acceleration A is, in essence, a roundabout way to obtain the effective acceleration without analyzing real values of a_{max} and, in addition, to eliminate the majority of the aforementioned difficult problems involving the relationships between I_d and a_{max}, as well as between a_{max} and A. Consequently, the very concept of using the it as a GSR map parameter, complemented with an explicit specification of the parameter $A(I_d)$ as an effective acceleration value, appears to be quite acceptable. However, in my opinion, the direct mapping of the parameter A is preferable.

A very serious problem relates to the reliability of design acceleration values, i.e., the correctness of the constant chosen for normative dependence $A(I_d)$. In order to exclude any bias in design loads, the relation $A(I_d)$ must be consistent with the average relationship between the observed values of a_{max} and I. The problem is that, as the number of recorded accelerograms rapidly increased over the past decades, the observed average of a_{max} corresponding to a fixed intensity also increased. The intensity $I = 7$ (9) corresponds to $A = 0.1$ (0.4) in [*SNiP II-7-81*, 1982; *SNiP II-7-81**, 2000]. On the other hand, the most recent experimental estimation from U.S. data [Wald *et al.*, 1999] yields an expected geometric mean of $a_{max} = 0.25\ g$ ($0.90\ g$) for $I = 7$ (9), which amounts to 220–250% of the values adopted in the current SNiP code. This implies serious problems that are discussed below.

Here, I discuss alternative approaches. The use of amplitudes, rather than intensities, in GSR maps automatically removes the problem involving the relation $A(I_d)$, and this is an obvious advantage of such an approach. On the other hand, the problem of conversion to effective amplitudes discussed above becomes more stringent. I start with the case when a GSR map shows design values of the peak acceleration (A). In an extreme case, seismologists constructing the map can ignore the ideology of an effective acceleration that is not generally acknowledged as yet and can thereby map an averaged real "seismological" value of a_{max}, neglecting the problems of isolated or short impulses, peculiar spectra, and duration effects of ground motions. Then, in using the map, one can obtain erroneous results. In particular, an overestimated engineering hazard can be ascribed to territories where the seismic hazard is dominated by small-magnitude sources, and vice versa, an engineering hazard can be underestimated in areas dominated by large magnitudes (because long duration effects of accumulating defects are ignored). In another extreme case, engineers estimate the effective acceleration A by combining seismological data and engineering considerations and map A values precisely; as a result, the map loses its direct seismological meaning (as compared with a map showing peak accelerations with a given recurrence). This contradiction could be resolved by a formal definition of the effective acceleration. Then, two independent variants of maps showing, respectively, seismological and effective accelerations could be constructed.

An *RA* map completely removes the problems of isolated and short impulses and unusual spectra; the problem of the duration effect incorporation is also solved in part (only for short accelerograms). The only remaining problem is the incorporation of the duration effect in the case of earthquakes with magnitudes of 7.5–8.5 when the accumulation effects of defects can be strongest. This disadvantage is essentially inevitable, if the method of response spectra is applied in line with the concept of a linear time-invariant system that is basically incapable of accumulating defects. However, this difficulty can apparently be overcome by using a duration-dependent correction. Following the arguments presented in [Aptikaev, 1975; Aptikaev and Shebalin, 1988; Kennedy, 1980], the correction to the spectral amplitude can be represented as

$$RA(T)/RA_0(T) = (d/d_0(T))^{\varepsilon},$$

where $RA(T)$ is the recommended design value, $RA_0(T)$ is the preliminary value of the response spectrum calculated by the ordinary method, d is the duration of a design (scenario) earthquake, $d_0(T)$ is the time over which the oscillator with a period T and a damping D attains the stationary regime ($d_0(T) \approx 2T/D$, which yields $40T$ at $D = 0.05$), and ε is an empirical factor close to 0.15.

Summarizing, I can state that the argumentation adopted in Russia for the GSR map construction in terms of intensity is acceptable from the conceptual point of view. In essence, this is a means for introducing an actual and reasonable definition of the effective design acceleration A. If the accepted relation $A(I_d)$ is authentic, this type of map can remain valid in future, although the direct use of the design effective acceleration as a map parameter appears to be more consistent.

However, the present, more than twofold divergence between the dependence $A(I_d)$ adopted in [*SNiP II-7-81**, 2000] and data derived from the analysis of a large body of strong motion observations is a very serious well-known problem. As seen from the table, the value $a_{max} = 0.4\ g$ adopted in [*SNiP II-7-81*, 1982] for an intensity of 9 was, in a sense, correct because it overlaps from above the respective range of the MSK-64 scale (0.2–0.4 g). However, the table also shows that accumulating data changed these estimates. By 1981, a value of 0.4 g was actually obsolete, even as an average estimate. Aptikaev and Shebalin [1988] recommend an estimate of 0.60 g for an intensity of 9. The new estimate of Wald *et al.* [1999] is already as high as 0.92 g. However, I should note that both these estimates are, to an extent, artificial. The acceleration actually stops increasing near an intensity of 9 (the a_{max} parameter is saturated). The empirical average for an intensity of 9 is 0.45 g according to Aptikaev and Shebalin [1988]

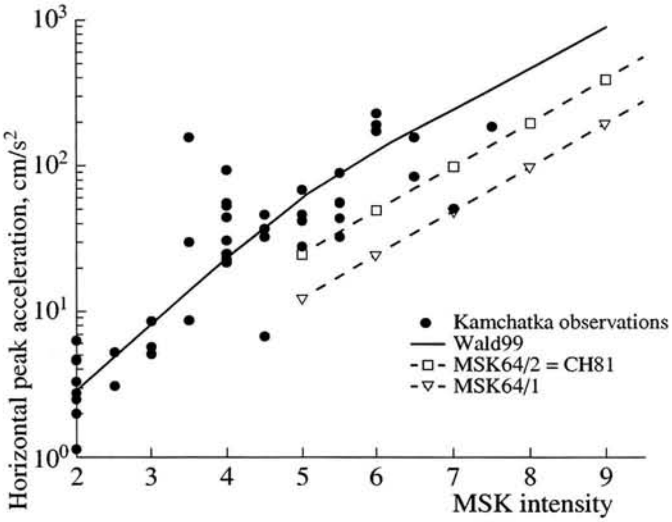

Fig. 1. A preliminary variant of the dependence of observed horizontal peak accelerations in Kamchatka on the MSK-64 seismic intensity. Triangles and squares are lower and upper bounds of the acceleration range for intensities from the amplitude part of the MSK-64 scale [Medvedev, 1968]. The solid line is the dependence after Wald *et al.* [1999].

and about 0.5 *g* according to Wald *et al.* [1999]. However, this saturation of the $a_{\max}$ parameter is insignificant with regards to the engineering design.

The point is that the $a_{\max}$ saturation can by no means be regarded as evidence of the destructive effect of an earthquake (otherwise, the intensity would stop increasing). In particular, the observed maximum velocity of ground motions continues to increase, showing no evidence of saturation. (A separate analysis of different types of ground might reveal a tendency toward saturation in weak ground, but to the best of my knowledge, such an analysis has not been conducted.) A stop in the $a_{\max}$ increase appears to be related to a decrease in the characteristic frequency of the ground motion spectrum with increasing intensity [Chernov, 1989; Chernov and Sokolov, 1988]. This tendency is due to the joint action of at least two factors: an increase in the contribution of a relatively low frequency nonwave component of the ground motion near a fault and a rapid increase of nonlinear losses in ground with increasing amplitude discussed below. On the whole, the speculative effective parameter *A* must undoubtedly increase with intensity.

It is widely acknowledged that data from the United States, where the modified Mercalli scale [Wood and Neumann, 1931] is used for estimating the macroseismic intensity, are somewhat incomparable with the intensities of European scales. I started to verify the results of Wald *et al.* [1999] with data of Russian earthquakes (Fig. 1). These results are seen to be verified in the representative range of data. Actually, the amplitude part of the MSK-64 scale seems to significantly underestimate the real acceleration parameters.

I note incidentally that the SNiP II-7-81* value $A = 0.1 \times 2^{I_d - 7}$ is also used as a recommended lower limit of the peak acceleration for the accelerogram employed in the numerical structural design. This is reasonable, but typical realistic peak accelerations are evidently twice as large.

WHICH GROUND SHOULD BE CHARACTERIZED BY THE INTENSITY SHOWN IN A GSR MAP?

The tradition to refer the GSR map data to a medium ground (category II according to the SNiP regulations) is well understandable: territories of the preferable bedrock ground (category I) are few, and to take a territory of the unfavorable soft ground (category III) as a reference basis is inconvenient. However, the adequacy of this approach raised doubts when it became clear that the high-intensity mechanism of the effect of ground conditions on the damage extent of structures qualitatively differs from that of low intensities. Imagine an Earth-surface area including sites of grounds of the three categories, and assume that this area is subjected to the effect of earthquakes of various strengths and that this effect is homogeneous in the sense that waves striking the geological bedrock basement in this area are similar in amplitude, duration, and spectrum. Then, macroseismic effects (primarily, building damage) and thereby the observed intensity should increase approximately in proportion to the category of ground. Such behavior at lower intensities is primarily related to the fact that an increase in the ground category decreases the impedance (acoustic rigidity, ρV_s) of the upper ground layer, and accordingly, ground motion amplitudes (at least, velocity amplitudes) increase. This is a simple linear scheme of harmonic ground motion [Medvedev, 1962]. At higher intensities (8–9 and more on a bedrock ground), a nonbedrock ground fails under the action of seismic waves (ductile behavior and ground flow, fractures, pseudofluidization, and similar effect arise); as a result, basically different behavior patterns are activated. On the other hand, a fractured ground loses its bearing capacity and cannot support buildings and structures. On the other hand, a high-amplitude wave cannot travel through a weak ground, because the weaker the ground, the lower the maximum amplitude of waves that can be observed on its surface. Thus, at higher intensities, the wave amplitude drops with increasing ground category, and such behavior drastically differs from what is observed at lower intensities. On the other hand, the destructive effect of an earthquake increases with the ground category, but this is related not to the rise in the seismic wave amplitude but to the increasing degradation of the bearing capacity of ground and primarily to the nonuniform subsidence of ground under a building.

On a qualitative level, the picture described above was reported to the author by N.V. Shebalin in 1980.

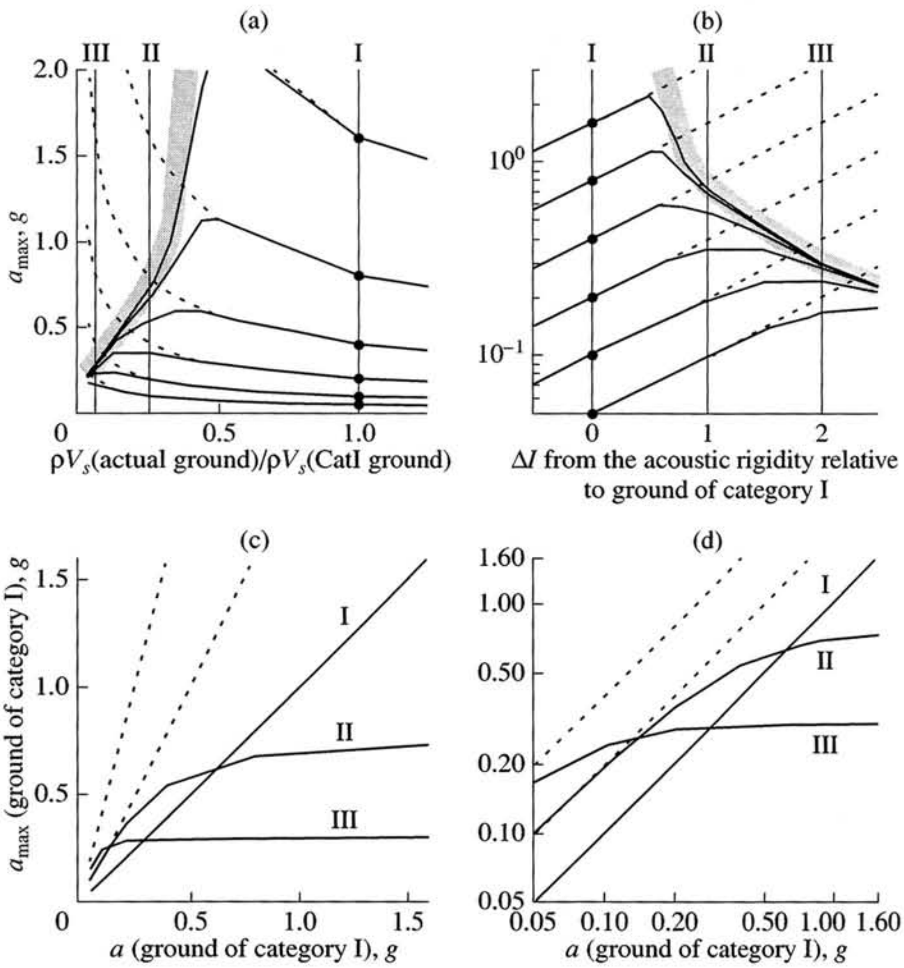

Fig. 2. A tentative scheme illustrating the effects of nonlinearity and ground failure on the peak acceleration at the ground surface. (a) Dependences of the ground surface peak accelerations on the normalized acoustic rigidity of the upper layer ρV_s for various amplitudes of a wave incident from below. Broken lines show linear behavior of amplitudes, and solid lines show their actual behavior. The shaded band shows the ultimate strength of ground expressed through the maximum seismic acceleration. Vertical lines II and III relate to grounds of the respective categories (medium and soft). (b) An analogue of plot (a) on a log–log scale, with the abscissa axis plotting the intensity increment ΔI relative to the ground of category I. (c) The peak acceleration a_{max} on grounds II and III as a function of the a_{max} value on ground of category I. Each line in this figure corresponds to the set of points on the respective vertical line in plots (a) and (b). (d) Curves (c) plotted on a log–log scale.

Several macroseismic observations supporting this point of view have long since become classics of seismology (the Niigata earthquake of 1964) or were even included into the text of macroseismic scales (fissures in ground and landslides). Particularly convincing are data of simultaneous recording of strong motions in a borehole and on the surface.

The phenomenon discussed is very complicated, and many of its detail are unclear, but its essence is as follows. With increasing amplitude of the seismic wave striking a ground layer, the elastic strain in the wave ultimately exceeds a critical value of the ground failure. Instrumental observations can fix even early fracture stages, whereas visual manifestations become evident at high amplitudes. This factor leads to the destruction of buildings and surface structures due to the nonuniform subsidence of basements (rather than the usual lateral force mechanism of fracture). If the subsidence is uniform, a structure can simply sink (without destruction) into a fluidized ground. Note in this connection that the possible destruction of a structure due to nonuniform deformations at its base is not provided for at all by the building code as an important design aspect.

The effective propagation of a wave in a failing ground is impossible (its energy is expended on the fracture). If the amplitude of a wave incident from below rises, its amplitude above the fractured layer will remain, very roughly, constant. Actually the amplitude can weakly increase because the ground degradation decreases the effective shear modulus, as well as the effective impedance (acoustic rigidity), but this effect plays a secondary role. I emphasize that effect of slowing down and saturation of the increase in amplitudes nonbedrock ground motions and their increase on a bedrock basement is not an anomaly: it is a universal phenomenon typical of a strong earthquake. This effect is noticeable (even at 0.2–0.3 g) primarily in grounds of category III, but with increasing amplitudes of ground motions in the geological bedrock basement, it is also observable on medium grounds. Mohammadioun and Pecker [1984] made a paradoxical statement that a_{max} on a nonbedrock ground can never exceed an average level of 0.5 g; this statement has a trustworthy observational basis (see acceleration values in the table). One can roughly set the ultimate acceleration at 0.25–0.4 g for category III and 0.5–1.0 g for category II, whereas the acceleration of a bedrock ground (or a tentative acceleration of a geological bedrock basement) can reach 1.5–2 g and more. Thus, with increasing ground motion amplitudes of the geological bedrock basement, the damage extent (and intensity) on the surface of a nonbedrock ground rises, and amplitudes of both acceleration and velocity stop increasing (become "saturated").

The important phenomenon discussed here is nonlinear, and Fig. 2 qualitatively illustrates the effect of nonlinearity and ground failure on the surface peak acceleration. Two upper plots (Figs. 2a and 2b) present two variants of the peak acceleration behavior on the ground surface as a function of properties of the upper ground layer. Such a property in Fig. 2a is the acoustic rigidity of this layer ρV_s. The curves are constructed for several amplitudes of a wave incident from below marked by solid circles on the vertical line indicating a typical ground of category I (bedrock). The circles indicate the following peak accelerations on such a ground: 0.025, 0.05, 0.1, 0.2, 0.4, 0.8, and 1.6 g. The broken lines are amplitude curves constructed under the assumption of linear behavior of the ground (infinitely strong ground). The vertical lines indicate ground categories II (medium-density ground) and III (soft ground). The acoustic rigidity values are normalized to the acoustic rigidity of ground category I. Intensity increments ΔI_{ar} relative to a reference ground of category I are plotted on the abscissa axis in Fig. 2b; the increments are calculated, as usual, through the ratio of acoustic rigidities:

$$\Delta I_{ar} = -1.66\log(\rho V_s(\text{actual ground})/\rho V_s(\text{reference ground})).$$

The amplitudes are plotted on a logarithmic scale. Using the same numerical data, Fig. 2c plots the peak acceleration a_{max} on surfaces of grounds of categories II and III as a function of a_{max} on the category-III ground surface. Figure 2d shows the same plots on a log-log scale. The plots in these figures are constructed from the points on the two vertical lines in each of the upper plots. The ultimate acceleration dependence on the acoustic rigidity accepted in constructing these plots and the related absolute values of ultimate accelerations on grounds of categories II and III (equal to 0.3 and 0.7 g) are purely tentative.

Figures 2a and 2b show that, with increasing amplitude of the wave incident from below, the amplitude on the surface first proportionally rises and then approaches an ultimate level, which is lower for weaker grounds. Furthermore, whereas at small amplitudes of the wave incident from below the intensity increments in acoustic rigidities successfully predict the amplitude behavior on the surface, higher incident wave amplitudes give an opposite pattern: the surface amplitude drops with a rise in ΔI_{ar}, as is clearly seen in Figs. 2c and 2d, where the incident wave amplitude is laid off on the abscissa axis. Based on the Ramberg–Osgood model of the medium, Mohammadioun and Pecker [1984] presented an example of the nonlinear behavior calculation of a soft ground layer under the action of a high-amplitude seismic wave incident from below.

However, due to the broadband nature of the seismic signal and the attenuation effects, the relationship between the amplitude and ground category is not trivial even in the case of linear ground behavior (at low and moderate intensities). Now I return to the case of three sites differing in the ground category located in one area (see above). The earthquake effects and the observed intensity increase approximately in proportion to the category number (from I to III). However, even at low intensities, accelerographs will not record any significant rise in the acceleration amplitudes. The virtual insensitivity of peak accelerations to the ground category is a reliably established and well-known fact (e.g., see [Campbell, 1988; Joyner and Fumal, 1985]). This is mainly due to the fact that the peak acceleration is associated with a high-frequency spectral interval (4–10 Hz, or 0.1–0.25 s), where the ground absorption is usually high. The absorption-related drop in high-frequency amplitudes tends to largely compensate their rise due to a small impedance; as a result, acceleration amplitudes weakly increase with the ground category even at low intensities. Nonlinear effects arise at intensities of 7–8, and this weak increase first stops and is then replaced by a decrease due to the change in the amplitude behavior pattern mentioned above. Such effects are averaged in the process of extensive processing of strong motion records obtained at intensities of 5 to 9, and the smoothed empirical pattern results in a virtual insensitivity of the peak acceleration to the ground category. Of course, these experimental facts strongly contradict the twofold increase in $A(\Delta I)$ per unity intensity implicitly accepted in the current SNiP II-7-81* regulations (see the table).

Nevertheless, a rise in the design acceleration A reflects an actual increase in the damage extent with the ground category. The point is that this dependence is controlled by factors other than the real peak acceleration of ground. In particular, I address the response spectrum. Its dependence on the ground type yields a small or negligible RA increment with the ground category in a range of 4–10 Hz (due to the same factors as those in the case of the peak acceleration discussed above). On the other hand, *RA* steadily increases with the ground category in the range of 0.5–2 Hz (this behavior is also characteristic of the maximum velocity). Intermediate effects are observed between these frequency ranges. Therefore, both velocity amplitudes and building damage are controlled by the ground type at relatively large periods. This is linear behavior associated with typical transfer functions of soil beds (spectral corrections for ground); it is generally independent of the ground motion amplitude and intensity. Thus, the ground effect basically depends on ground motion frequency. If it is adopted to be frequency independent, certain errors arise. Unfortunately, these (systematic) errors lead in some cases to substantial underestimates of design loads and the related decrease in the design earthquake resistance. More specifically, this situation is typical of buildings having small natural periods. This problem is discussed in detail below. The strongest dependence of the ground effect on frequency is well illustrated in [Okamoto, 1980, Fig. 5.14].

Note that, at small amplitudes (below a peak acceleration level of 0.2 *g*), the plots shown in Figs. 2c and 2d are poorly consistent with the idea of low sensitivity of the acceleration amplitude to the ground properties advocated above. The contradiction is undoubted, and it seems preferable to plot, for example, the peak velocity on the vertical axis in Fig. 2 (this is the reason why the plots in this figure are largely illustrative). The choice of the peak acceleration as an amplitude parameter in Fig. 2 is related to the fact that literature data provide constraints on the tentative realistic saturation levels only for acceleration amplitudes (accepted here equal to 0.3 *g* for ground category III and 0.7 *g* for ground category II). No adequate estimates for upper limits of other parameters (e.g., maximum velocity) could be derived from literature data.

The above facts are important for many reasons. Within the framework of the present section subject, they imply that a direct distinct relationship between intensity and amplitude exists at both low and high intensities only for a strong bedrock ground with weak nonlinear effects. Consequently, the normative GSR map parameters should relate to a bedrock ground representing the simplest and clear case. This requires an appropriate correction of historical macroseismic data and the intensity–magnitude–distance relationship employed in the construction of such a map or its verification.

SHOULD THE GROUND EFFECT BE EXPRESSED BY INTENSITY UNITS?

The tradition of expressing the ground effect through intensity units (in the form of the intensity increment ΔI) has long been established. In the simplest case, the incorporation of ground conditions involves the following assumptions [Medvedev, 1962].

(1) The ground effect is the effect of a rise in amplitude parameters of ground motion.

(2) The rise in the amplitude of the velocity V is determined by the energy conservation during the wave propagation (the energy density is $P = c\rho V^2$ so that $V_1 = V_0(c_0\rho_0/c_1\rho_1)^{0.5}$ and therefore $\Delta I_{01} = 1.66\log(c_0\rho_0/c_1\rho_1)$, where V_0 and $c_0\rho_0$ relate to a certain reference (e.g. medium) ground, and V_1 and $c_1\rho_1$, to the ground under consideration.

(3) The behavior of the acceleration amplitude is similar to that of the velocity amplitude.

Such is the argumentation underlying the SNiP regulations of the repetitive housing design: ground lithology evidence is used for determining ΔI_{01} (for example, $\Delta I_{01} = -1$ for a high-$c\rho$ bedrock ground). In this connection, it is important to note that the parameter $c_0\rho_0$ characterizes the upper 10 m of ground and that the effects of relief and adjacent faults are formally unaccounted for. If a designer requests seismic microregionalization (SMR), the so-called instrumental intensity increment is used instead of the quantity defined in point 2. It is calculated, in accordance with the norms presented in [*RSN 60-86* ..., 1986; *RSN 65-87* ..., 1987], by the formula

$$\Delta I_{01} = 1.66\log(x_0/x_1),$$

where x is an amplitude parameter (peak acceleration, velocity, or even displacement, response spectrum, or Fourier spectrum); the indexes 1 and 0 refer, respectively, to the design and reference site ground; and measurements are made, for example, during a weak earthquake. Evidently, the documents RSN60-86 and RSN65-87 reflect the pre-computer epoch, when the broadband nature of ground motions was not fully appreciated and the primitive idea that displacements, velocities, and accelerations behave in a similar way was believed to be valid. The spectral approach was not systematically applied in these documents, in part because it yields, roughly speaking, a ΔI value specific of each frequency band. These (normative!) documents give no recommendations as to what to do if the ratio of spectra obtained from the design and reference grounds nearly always significantly varies with frequency.

Presently, the spectral approach based on the digital processing of low-amplitude ground motions is widely applied in SMR investigations. Therefore, the idea of a general coefficient (a factor of 2 per unity intensity) common to displacements, velocities, and accelerations is absolutely unacceptable in SMR research. This is basically due to the fact that in some cases the current practice automatically leads to dramatic underestimation of design loads.

This important statement needs be verified, which can be done simply. I proceed from the principle according to which the design load should be proportional to the real seismic load. The SNiP value $\Delta I = -1$

adopted for a bedrock ground (category I) means a decrease in the design load by about two times relative to the GSR-reference medium ground (because the normative curves $\beta_{\Delta I}(T)$ weakly depend on the ground type). However, as is evident from the detailed discussion in the preceding section, real ground motions on bedrock and medium grounds have similar accelerations at intensities of 8–9. Then, if the controlling scale parameter of ground motions is the tentative peak acceleration A, this parameter should be assumed to be nearly or fully independent of ground conditions. Because its normative value is twice as small, the 4–5 Hz loads are significantly underestimated (by up to two times). Natural frequencies of about 4 Hz are typical of ordinary one-story houses. The design load for these buildings will be underestimated by about two times.

I note once more that this fact is consistent with variations in the typical average level of damage generalized over various construction types. Although the peak acceleration is approximately constant, the response spectrum form is usually strongly dependent on the ground type and provides both appreciably different velocities (about two times per unity intensity) and the respective differences between loads for all structures having natural frequencies of 2.5–3 Hz and less. As a result, the SNiP II-7-81* scheme incorporating the effect of ground conditions by A variations yields a correct result for a bedrock ground at frequencies below 3 Hz. With the criticism presented above, such a good result seems strange; actually, this is only due to the fact that the shape of the SNiP II-7-81* plots $\beta_{\Delta I}(T)$ (identical plots for categories I and II with a somewhat different plot for category III) was selected, to a large extent, artificially with the intention to ensure this result. The insensitivity of these plots to the ground type is poorly consistent with seismological reality.

The resulting conclusions are as follows.

A. On the Method Incorporating the Effect of Ground Conditions

(1) Because the ground effect is frequency-dependent, the use of the intensity increment ΔI is unsuitable. Using the engineering spectral method, this effect can be appropriately accounted for by the frequency-dependent increment in the logarithm of the response spectrum (ILRS) $\Delta \log RA(f)$ (or the numerically similar increment in the logarithm of a smoothed amplitude Fourier spectrum (ILFS) $\Delta \log a(f)$).

(2) The determination of the frequency-dependent ILRS relative to the bedrock/dense ground (category I) should be considered as a main task of SMR.

(3) The instrumental SMR should include an extensive use of the reference bedrock ground or special studies for determining seismological characteristics of the reference ground/site accepted.

(4) If the spectral analysis cannot be conducted in the instrumental SMR, one should assume that the ILRS is determined by the ratio of velocity amplitudes. In using accelerations, serious errors can be expected. The resulting values of the increment in the logarithm of velocity amplitudes should be regarded as ILRS estimates in the frequency band corresponding to the maximum of the strong motion velocity spectrum (usually 0.5–1.5 Hz).

In extreme cases, these estimates can be extended to other frequencies. This can be done by extrapolation based on typical curves ILRS(f) previously obtained from detailed studies of ground layers that can serve as an analogue of the ground in question.

(5) Along with the spectral analysis, the SMR should include the calculation of frequency-dependent ILRS based on the spectral-response ratio estimated either directly or from the ratio of smoothed amplitude Fourier spectra.

(6) In estimating the ground effect from lithology without seismological evidence, one can believe, following Medvedev, that the ILRS is determined by lithology (via the impedance ratio). However, the use of parameters of a thin (10 m) layer is precarious (such a thickness is commonly less than a quarter of a wavelength). For engineering purposes, one can use (following the current practice in the United States), for example, parameters of a 30-m thick layer. The resulting ILRS should then be applied as in point 4.

(7) Let future norms, like the present ones, distinguish three types of ground effects corresponding to categories I, II, and III (henceforth referred to through indexes CatI, CatII, and CatIII). In the approach proposed, the design response spectrum should then include, generally speaking, three factors: (1) the A value for the bedrock ground in the site area (e.g., obtained from (($I_{d0}-1$) by a realistic formula of the type $A = 0.2 \times 2^{I_d-7}$); (2) $\beta(T)$ for the reference bedrock ground (denoted as $\beta_{\mathrm{CatI}}(T)$); and (3) ILRS(T) for the actual ground relative to the reference ground. I consider two specific cases.

(7.1) The designer knows the lithology alone. In this case, the norms should include either the standard functions

$$\beta^*(T|\mathrm{CatII}, \mathrm{CatIII}) = \beta_{\mathrm{CatI}}(T)10^{\mathrm{ILRS}(T|\mathrm{CatII},\,\mathrm{CatIII})},$$

determining the total effect of the last two factors or, which is the same, the standard ground corrections ILRS(T|CatII and CatIII). In an ideal case, the functions $\beta^*(T|\mathrm{CatI}, \mathrm{CatII}, \mathrm{CatIII})$ should characterize typical or general combinations of spectral properties of earthquakes and prevailing grounds in a concrete region (as a last resort, they can be general for the entire country).

(7.2) The SMR is accomplished by a seismological or other similar method so that the experimental value

ILRS($T|_{exp}$) is known. All three factors are then used separately. The product $A\beta_{CatI}(T)$ gives the design value $RA(T)$ for the reference ground of category I, and the coefficient $10^{ILRS(T|exp)}$ corrects the result for the ground of a specific site. This scheme need be slightly complemented for a nonrocky reference ground.

The scheme described above basically differs from the present one, when the calculation is based on the A value characterizing a given site rather than a hypothetical site composed of a bedrock ground, and the ground effect is incorporated through the parameter ΔI, which is precisely what I propose to reject. (Note that my proposals can be easily formulated in terms of the frequency-dependent increment of intensity $\Delta I(f) = \mathrm{ILRS}(f)/\log 2$, but the introduction of such an artificial construction is, in my opinion, very improper.)

B. On the Underestimation of the Seismic Hazard in the Current SNiP II-7-81 Code*

The incorporation of the ground effect through the intensity increment in combination with the adopted shapes of the curves $\beta(T)$ results in a nearly twofold underestimation of design loads for small-scale structures on grounds of category I and thereby leads to a significant relative decrease in the safety of inhabitants of one- and two-story buildings constructed in accordance with the regulations.

ON THE INTENSITY ROUNDOFF

The current tradition of using only rounded, whole-number values of I_{d0} and ΔI is evidently inadequate to reality. As a matter of fact, both the calculation of shaking capacity and the estimation of the ground conditions effect (through the impedance or experimentally) yield continuously varying numerical results. In the context of the norms, the intensity is a conditional value functionally related to the design acceleration, and there are no basic reasons to believe that it changes in a discrete manner. The I_{d0} and ΔI roundoff procedure produces errors reaching ± 0.5. The error of the sum of these values attains ± 1, which amounts to a twofold error in the amplitude. This error appears as a random error only from the viewpoint of Moscow; for inhabitants of a concrete settlement (or a site within), it means a permanent under- or overestimation of the design load by a factor of up to 2. The reason why this principle is preserved in current practice is absolutely unclear, particularly if one remembers the general principle of numerical calculations, namely, to retain the additional significant digit in intermediate calculations. The interpolation of the GSR map or introduction of zones of fractional intensity (7.0, 7.25, 7.5, and so on) are not difficult, and such a detailed approach is quite correct from the standpoint of the inner accuracy of calculations involved in the construction of the map. An illustrative example is presented in [Gusev, 1991]. The calculation of ΔI from lithological evidence and particularly from seismological data allows one estimate ΔI with an accuracy of 0.2–0.4 rather than 1.0. The further preservation of the whole-number rule in estimating I_{d0} and ΔI is an unnecessary and injurious conservatism.

The rule of whole-number design intensity has led to specific consequences in the new situation when the SNiP II-7-81* norms (as distinct from the previous SNiP II-7-81 norms) contain three (rather than one) GSR maps related to three recurrence periods (500, 1000, and 5000 years for maps A, B, and C, respectively). Each point of the territory is now specified by three values of I_{d0}. For simplicity, I introduce the parameter of intensity increment over the recurrence period $\Delta I_{T_1/T_2}$ as the difference between the design intensities in the maps of the periods T_1 and T_2. Since each of the three maps is discrete, the value of $\Delta I_{T_1/T_2}$ at each concrete point is also discrete. For example, the difference $\Delta I_{1000/500}$ assumes values of 0 and 1 with comparable probabilities. I remind the reader that the creation of the set of maps aimed to account for the significance level of structures: map A provides design loads for the majority of buildings, and map B, for vital structures (airports, hospitals, and so on). Map C is specially commented below. Due to the discreteness of $\Delta I_{1000/500}$, we have in practice that the design loads calculated for vital structures either do not account for their significance or provide an overestimated coefficient. Actually, this is a simple consequence of the roundoff procedure, but it leads to confusion in understanding the matter.

An alternative way is possible. As shown in [Gusev, 1991], a 1.00 increment in the nonrounded design intensity (more specifically, the inverse shaking capacity) due to variations in the recurrence period correspond to an increase in the recurrence period by a factor of 5–10. Then, a twofold increase in the period (from 500 to 1000 years) should yield an intensity increment of 0.30–0.43, and the design load will increase, respectively, by a factor of 1.23–1.38. An immediate check on the basis of data used for construction of the OSR-97 maps showed that region-averaged intensity increments ($\Delta I_{1000/500}$) lie within a 0.2–0.7 range (a load variation within 1.15–1.62 times). Although the incorporation of areal variations in this parameter is not altogether meaningless, it is largely unjustified against roundoff errors. It seems much more natural to neglect, in the engineering approximation, such variations and to account for the significance degree of a structure by a general increasing load factor of about 1.4–1.5. (A similar coefficient in the US regulations is 1.50.)

There exists another argument against the use of a recurrence greater than 1/500 in GSR maps. The point is that an increase in the recurrence period inevitably decreases the reliability of the design values of the shaking capacity. Thus, map A (500-year period) is no more than an expert generalization of seismological observations (mostly available over the last 50–100 years) involving a roughly tenfold extrapolation in time. The use of a constant coefficient responsible for the significance of structure removes the problem of a reliable further extrapolation, simply by rejecting the use of map B.

I note incidentally that the ambiguity of map C (a 5000-year recurrence period for strategic or dangerous structures) is *a fortiori* higher compared to map B. However, one should take into account that map C was primarily designed not as a simple normative map but as a map providing estimates for the planning of subsequent detailed investigations of the classes DSR and UIS.

ON LOADS AND DESIGN SCHEMES UNDER THE CONDITIONS OF A GROUND STABILITY LOSS

As mentioned above, seismological data indicate that the amplitudes of soft ground motions do not vary or even decrease with an increase in the motion amplitudes of the geological bedrock basement. However, in the absence of a noticeable rise in the amplitudes, the damage to buildings and, accordingly, the macroseismic intensity increase. We have a paradox: the damage increases, although the load is stabilized. This phenomenon is primarily related to the effects of a stability loss (failure and pseudofluidization) in grounds under buildings. (Given higher amplitudes, the same phenomena can also occur in medium grounds.) The calculation of similar loads is not provided for by the current norms [*SNiP II-7-81** ..., 2000]. However, according to previous norms, a designer had to increase by two times the design lateral load applied to a building on a ground of category III (relative to category II) in accordance with the macroseismic data and contrary to seismological data on amplitudes. A lateral load on a ground of category III, artificially overestimated by two times, induced the designer to reinforce the building as a whole, and this enhanced, albeit not very effectively, its resistance to a nonuniform subsidence of the underlying ground. Such an approach is in line with the general principle of increasing the safety factor under uncertain conditions. The building code in the new edition [*SNiP II-7-81** ..., 2000, point 2.5, note 2] reduces, in relation to the conditions described, the load to 70% of the initial value (apparently, taking into account seismological data on a relative decrease in amplitudes): a coefficient of 1.4, rather than 2, is proposed for use. Presently, the safety factor has become much lower, but this decrease is not compensated by realistic corrections for the nonuniformity of subsidences. Some techniques allowing for possible subsidences are described, for example, in [Okamoto, 1980, Section 7.5.2].

Not being a specialized engineer, I can only voice my apprehensions. In my opinion, the divergence between the intensity increase and stabilization of or drop in the acceleration amplitude on a ground of category III at intensities of 8 and more should be directly and explicitly allowed for. Namely, the calculation of the lateral inertial force should include its slowed-down and even zero-rate increase with the ground category with a simultaneous building design allowing for a nonuniform subsidence of the underlying ground. As a temporary measure, a factor of 2 used before 2000, rather than the new lower value 1.4, could be applied to the load on a ground of category III.

CONCLUSIONS

(1) The use of intensity as a building code tool is, in principle, no worse than the use of the peak acceleration and can even have certain advantages.

(2) The intensity–acceleration relationship $A = 0.1 \times 2^{I_d - 7}$, currently adopted in the SNiP II-7-81* code, is inconsistent with the average observed relationship, and this must substantially underestimate the actual design loads in Russia as compared with a competent seismological prediction. The load is shown to be underestimated by a factor of about 2.

(3) As a measure of the seismic hazard of a territory or a site, it is appropriate to use the design intensity on a ground of (bedrock) category I rather than (medium) category II, and the norms should provide for the transition from the category-I design intensity to the design response spectra $RA(T)$ on grounds of categories I, II, and III (or on grounds classified according to a more detailed scheme).

(4) A consistent use of the frequency-independent intensity increment is impossible in the spectral approach to the regulation of seismic loads. In particular, for this reason the current norms underestimate by about two times the correct design load for one-story buildings on a bedrock ground. The incorporation of the ground effect in terms of intensity increments should be replaced by the use of spectral (frequency-dependent) ground corrections to amplitudes.

(5) The principle of using solely rounded, whole-number intensities leads to artificial systematic errors in design loads reaching a value of 1.0 at some points and sites. Either a GSR map showing whole-number intensity contours should be interpolated, or a GSR contour map should be constructed, initially with an intensity step of 0.25. The same is true of ground corrections that should not be rounded at all or should be rounded to an intensity multiple of 0.25.

(6) The same whole-number principle leads to the fact that the difference between design intensities of the OSR-97-A and OSR-97-B maps at the same point is either zero or unity. As a result, the significance of a structure is allowed for in a seemingly nonsystematic way. A general increasing load factor of about 1.5 is proposed in order to allow for the structure significance, with the map A alone being used.

(7) An increase in damage (macroseismic intensity) appears to contradict a nonincreasing acceleration amplitude on a (soft) ground of category III at intensities of 8–9 and more. This inconsistency cannot be regarded as an adequate reason for a simple decrease in the design inertial load (lateral force) on such grounds. Such a decrease in the design load is justified only if a design for the structure resistance to a nonuniform ground subsidence is additionally worked out.

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