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Seismic design of Reinforced Earth[r] retaining walls – The contribution of finite elements analysis

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ABSTRACT : After experiments on reduced models and actual structures, dynamic finite elements analysis has recently been introduced into the continuing study of the response to seismic loads of Reinforced Earth[R] retaining walls. An overview is given of the results obtained at different levels of seismic excitation in a variety of foundation-soil conditions, and of their practical design implications.

1- INTRODUCTION

Being both heavy and flexible, Reinforced Earth[R] structures stand up well to vibrations and seismic loads. This has been demonstrated in reduced models, and in experimental structures subjected to strong vibration ; above all, though, it is confirmed by the excellent performance of the many structures - in Japan, Italy, New Zealand, USA, Mexico and even in Belgium - with in-service earthquake experience.

Most early research in this field was done in the USA by Lee and Richardson, in Japan by Uezawa and Chida, in New Zealand by Nagel and Elms and in France by Terre Armée.

The usual approach at this stage was to carry out tests on models built on shaking tables, or on full size walls to which strong vibrations were applied from above. In more recent years, however, in order to validate and further refine the calculation methods derived from such investigations, Terre Armée Internationale has been making increasing use of finite elements models.

2- EXPERIMENTS

2.1 Tests using laboratory models.

Tests on reduced scale models provide a good insight into the type of phenomena which occur ; quantitatively, however, they are much less informative, due to the problems of similitude.

30 cm high models, used for example by Richardson and Lee (1974), were built of sand stabilized with magnetic tapes 6 mm wide and 25 µm thick. The facings were made of aluminium elements 0,4 mm thick and 25 mm high. Vibrating table tests revealed two possible failure modes, similar to those found in static conditions : either failure occured fairly suddenly, as a result of strips breaking, or it was very gradual and due to insufficient adherence ; in the latter case, there was considerable associated deformation, but the wall recovered its equilibrium when vibration ceased.

2.2 Tests using a half size model.

Professor Chida's model (1980) was big enough to avoid the problem of similitude. As in actual applications, it had concrete facing panels, and steel strips. Its vibrating table allowed tests to be carried out at frequencies of between 2 and 7 Hz, for horizontal accelerations of between approximately 0,1 and 0,4g (Fig 1).

During the tests, measurements were made of dynamic increments in tension along the strips (Fig 2) ; these proved to be relatively evenly distributed, hardly changing the location of the point of maximum total tension (which governs the pull out resistance).

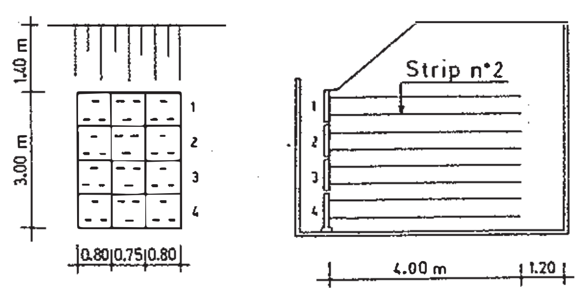

Fig.1 - Prof. Chida's half size model.

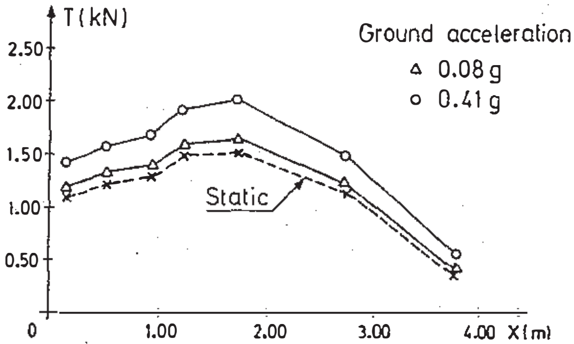

Fig.2 - Chida's test wall. Tension along strip n° 2.

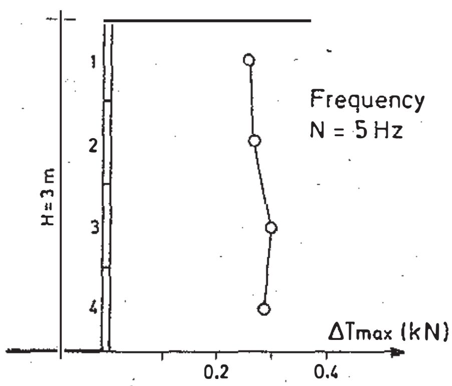

Fig.3 - Chida's test wall. Dynamic increments.

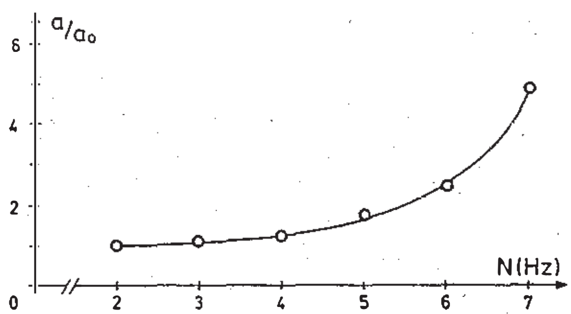

Fig 4 - Chida's test wall. Amplification of acceleration between base and top versus frequency.

At the same time, such dynamic increments seemed to vary very little with depth (Fig 3).

Professor Chida also measured amplification of horizontal acceleration as between the base and the top of the model. Relatively high values were found at higher frequencies ($a/a_0 = 2$ at 5 Hz), though this might well be largely due to the model's rigid frame (Fig 4).

2.3 Tests in actual structures

The tests undertaken by Terre Armée (France) in a 6 m high structure at Triel (1976) were in fact specifically aimed at evaluating transmission of vibration caused by a roller on top of the wall.

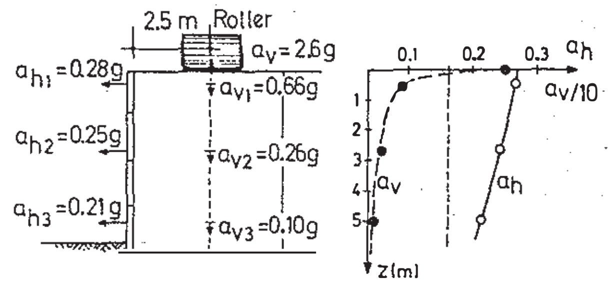

Fig.5 - Triel wall. Transmission of vibration.

Here, horizontal acceleration was found to vary little as a function of depth (Fig 5).

Meanwhile, in a general theoretical study commissioned by Terre Armée Internationale, Professors H.B. Seed and J.K. Mitchell estimated average amplification coefficients likely to be found in routine Reinforced Earth walls, as a result of their particular vibratory modes. A ratio of between 1,20 and 0,85 was suggested, for maximum accelerations at the wall base of 0,1 g to 0,5 g.

3- RESEARCH BASED ON FINITE ELEMENTS

3.1 Programme

Finite elements analyses had already been undertaken, on a considerable number of models, for a thorough investigation of the static behaviour of R.E. structures : Terre Armée Internationale's research in this direction (1982-84) relied on the Rosalie programme by LCPC (France). The dynamic study, on the other hand, was carried out using the SUPERFLUSH programme, a recent development, by Dr T. Udaka (Earthquake Engineering Technology, California), of LUSH, the programme created in 1974 by Lysmer, Udaka, Seed and Hwang.

Instead of rendering an accelerogram discrete in time steps, introducing instantaneous acceleration into the model for each step, Superflush uses a less costly approach, analysing the accelerogram in Fourier series per frequency steps. The elastoplastic behaviour of the soil is simulated by varying the modulus of elasticity as a function of observed deformation, this process being repeated until moduli and deformations are compatible.

The first Terre Armée model was gradually refined by, first of all, adjusting its results to fit those of Professor Chida's test wall. The close agreement between the experimental and the calculated results demonstrated the reliability of both the programme and the model (Fig 6). The representation of conditions encountered in actual structures was then improved, by introducing fictitious "half spaces" simulating the foundation soil and the rear backfill.

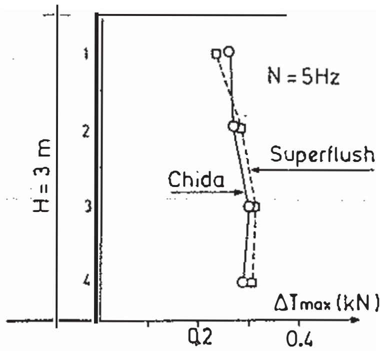

Fig. 6 - Agreement between Chida's measurements and Superflush results.

3.2 Models

The Reinforced Earth retaining walls represented in the last phase of the study (1987) are 6 m and 10,50 m high respectively, and similar (particularly as regards the length and number of their strips) to those most comprehensively analysed in the static finite element study (Fig. 7). The strip density in the 6 m wall is uniform throughout (4 40 X 5 strips per panel), whereas in the lowest third of the 10,50 m wall the strips are 1,5 times larger (4 60 X 5 strips per panel). The dimensions of the strips being those one would find in an actual structure, there is no risk of any soil / strip slippage.

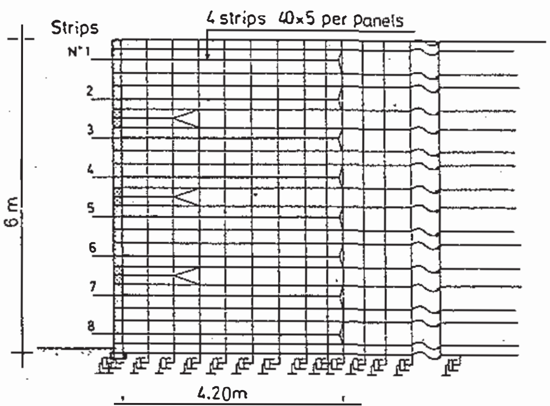

Fig.7 - F.E. model of the 6 m high wall.

These strips are in mild steel : obviously, the results of this research do not apply for other reinforcement materials having totally different moduli. The values assigned to the principal properties of the materials included in the models are summarized in the table below.

		Backfill	Steel	Concrete
Unit weight	pcf	125	460	150
	kN/m^3	19,65	72,28	23,57
Poisson's ratio		0,20	0,30	1/3
Shear modulus	ksf	$70\sqrt{\sigma m}$	1,661X10^6	2,050X10^5
	MPa		79,56X10^3	9,82X10^3
Damping ratio		$\sqrt{G/\rho}$	3%	7%

3.3 Environment and motions

The structures were placed on three types of foundation soil in order to analyse the influence exercised by local conditions. These soils are homogeneous and are distinguished by their shear wave velocity V :

- hard rock soil where V = 1000 m/sec
- medium stiff soil where V = 400 m/sec
- loose soil where V = 250 m/sec (due account being taken, in selecting this value, of the presence of the wall).

In addition, two different typical accelerograms were applied to the structures : first, that of the so-called Golden Gate S80E earthquake, which occurred in 1957 at San Francisco, and then that of the 1971 San Fernando earthquake, known as Castaic N21E. In the former there is a fairly marked predominating frequency around 8 Hz (Fig 8).

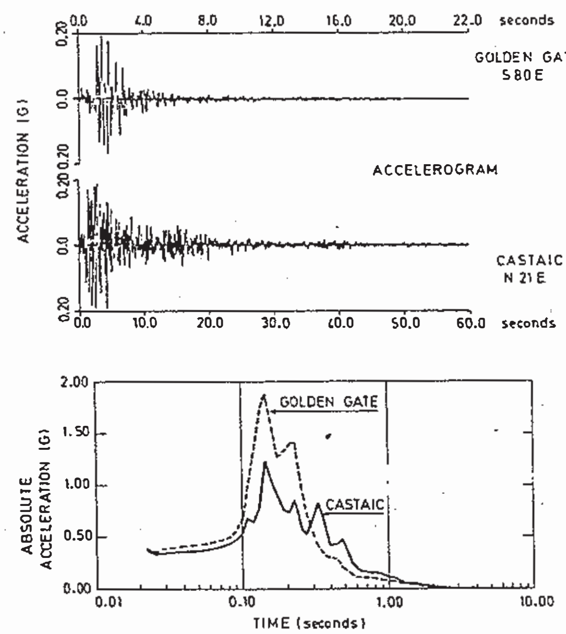

Fig.8 - Accelerograms and response spectra (top level).

And finally, each of these accelerograms was adjusted to fit several different maximum accelerations, chosen from amongst 0,1 g, 0,2 g and 0,4 g - the last of which in fact corresponds to a particularly severe earthquake.

3.4 Results

3.4.1 The influence of the environment

The results presented in graph form below are limited to those of particular comparative interest as regards maximum dynamic forces affecting the strips (which must,

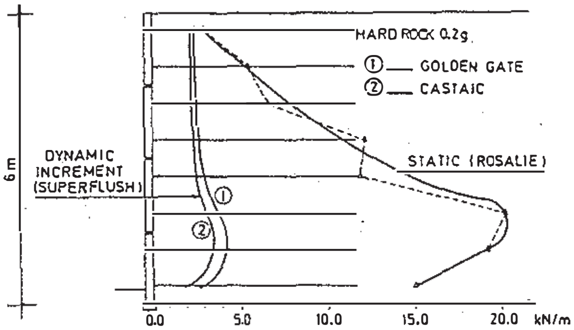

Fig. 9- Variation of maximum tension with depth. Influence of the type of accelerogram.

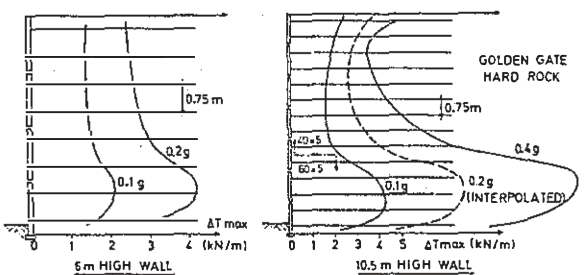

Fig. 10 - Influence of the maximum acceleration.

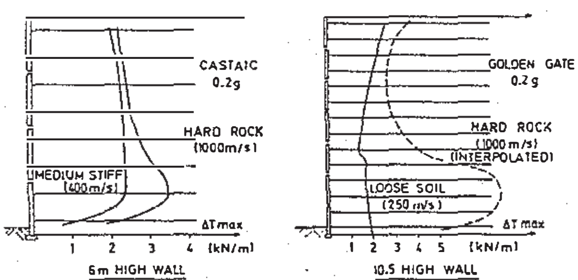

Fig.11 - Influence of the foundation soil stiffness.

of course, be added to forces of static origin).

On the same foundation soil (rocky) and at the same maximum acceleration (0,2g), the Golden Gate earthquake is a little more "aggressive" than Castaic ; this is not surprising, given their respective spectra, though the difference is really extremely small (Fig 9).

The structures respond much more sensitively to the maximum acceleration value of a single accelerogram, in respect of one and the same soil (Golden Gate / rock) (Fig 10)

There is also a very marked reduction in dynamic forces as a function of foundation soil stiffness, for a given seismic motion (Fig 11).

3.4.2 The distribution of dynamic forces

The finite element results confirm those obtained by Professor Chida : the distribution of dynamic tensile forces along the

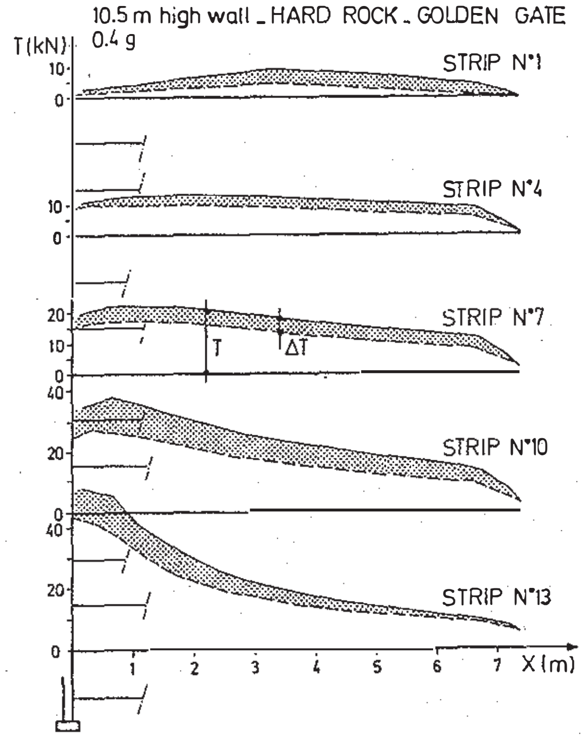

Fig. 12 - Superflush. Tensions along strips.

strips (or, to be more accurate, the envelope of dynamic tensile increments) is fairly uniform and, as can be seen in the graphs below, does not give rise to any significant change in the position of the points of maximum tension, even for strong accelerations (Fig 12).

In line, once again, with the Chida model findings, the maximum dynamic increment varies little with depth in structures reinforced in a uniform manner. There is, however, a tendency for lower strips to be relatively more affected, i.e. those strips having the longest resistant length (beyond the point of maximum tension). This result is a striking confirmation of what was forecast in 1981 by Professors H.B. Seed and J.K. Mitchell.

The tendency for lower strips to be more affected is particularly pronounced when they are wider or more numerous, as is the case at the bottom of the 10,50 m wall, i.e. when these strips have what might be termed a higher "bonding-stiffness".

3.4.3 Acceleration

The amplification of acceleration as between the base and the top of the model is by a factor of about 2, at least in the unfavourable case of structures sited on a hard rock foundation and subjected to the Golden Gate accelerogram. This ratio is quite comparable to those found experimentally by Chida. However, if an average value is calculated for the curves depicting variations in maximum acceleration over the height of the structure, and this average is set against the reference maximum acceleration at ground level, the increase is then found to be only by a factor of about 1,35 for 0,1g, and 0,95 for 0,4g (Fig 13). Such values, albeit somewhat higher, are nonetheless comparable to those suggested by Seed and Mitchell. Possibly the way in which boundary conditions are simulated in the finite elements model is still a little unfavourable ; on the other hand, if design calculations are indeed based on these values, any bias will be on the safe side.

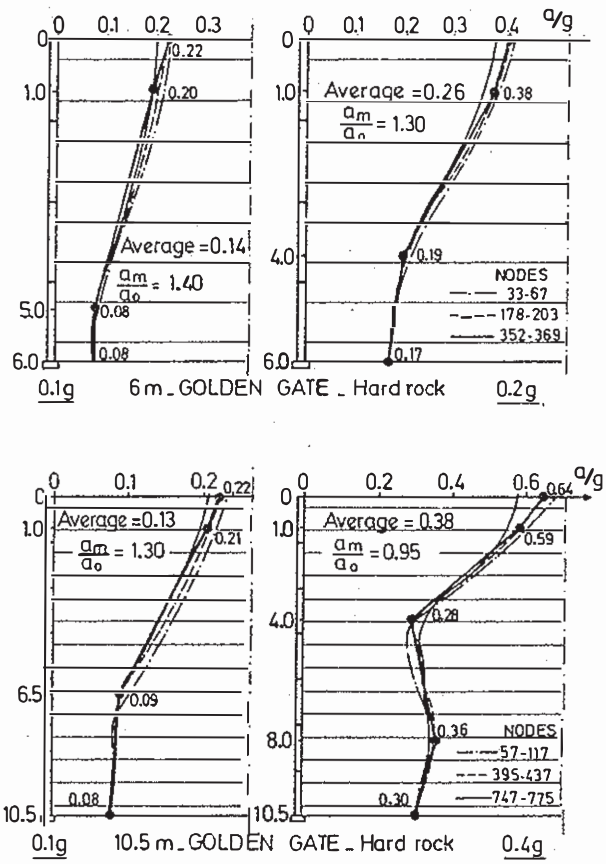

Fig.13 - Superflush. Amplification of acceleration.

4- PRACTICAL DESIGN METHOD

4.1 Internal stability

Analysis of these various results has led to the definition of a practical calculation method based on estimating a global inertia force E_I for the mass of the structure's active zone where the increments in tension occurring in the strips originate.

Given the envelope form adopted for the line of maximum tension in usual structures, we have (Fig 14) :

$$E_I = 0{,}20 \ \frac{a_m}{g} \gamma_1 H^2$$

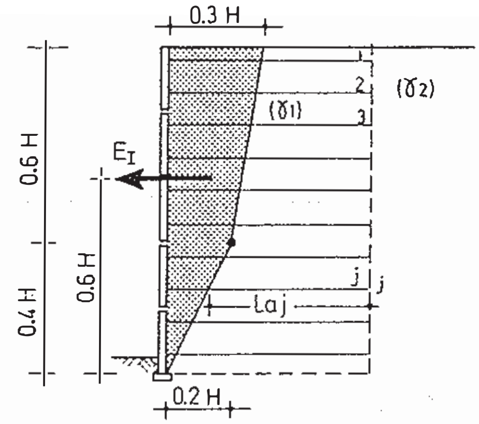

Fig.14 - Internal stability. Global inertia force E_I.

The value assigned to a_m is an estimation of the average maximum horizontal acceleration in the structure. In practice, for a reference acceleration a_o of between 0,05 and 0,5 g, it is assumed that :

$$\frac{a_m}{g} = \left(1{,}45 - \frac{a_o}{g}\right) \frac{a_o}{g}$$

for structures on rock foundations.

The load E_I is then distributed among the different layers of strips pro rata to their "resistant area" Nj bj Laj.

The graphs in figure 15 show that there is excellent agreement between the results achieved using these simple formulae and those calculated using finite elements.

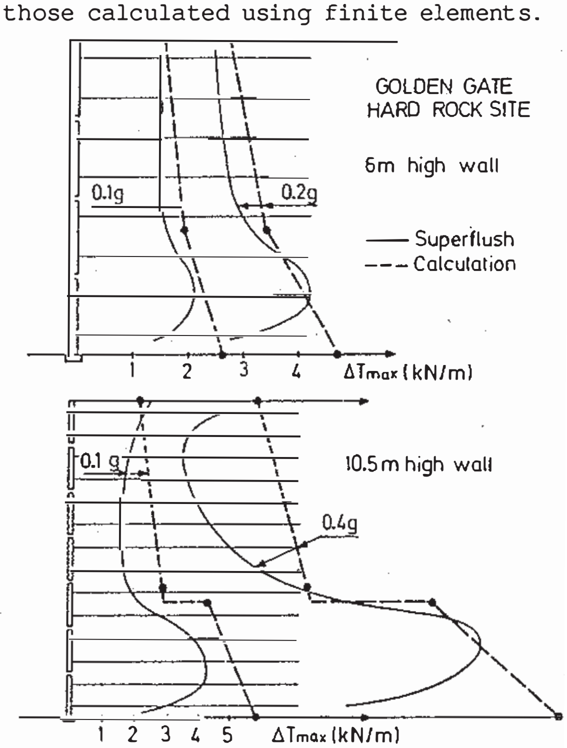

Fig.15 - Dynamic increments. Comparison between practical calculations and Superflush results.

These increments in tension are additional to static tensile loads resulting from the own weight of the structure as increased by the overturning effect due to the earth pressure of the backfill behind. This approach also includes a proportion of the supplementary dynamic (or pseudo-static) earthpressure E_{AE} calculated according to Mononobé Okabé (Fig. 16) (Taking only some 60% of dynamic pressure into account is justified by the fact that E_I and E_{AE} are unlikely to peak simultaneously).

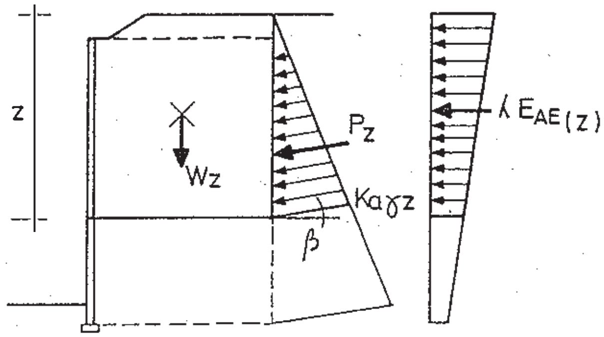

Fig.16 - Internal stability, External loads.

The strips are then checked in the usual way, applying the safety coefficients appropriate to accidental events such as earthquakes. When checking adherence, however, the apparent friction coefficient f* is limited to 80% of the value used in the static calculation.

4.2 External stability

When checking overall stability, the elements to be added to the structure's own weight and the static earthpressure of the soil to the rear are, this time, pseudo-static pressure E_{AE}, preponderant here, and a fraction (≈ 60%) of the structure's inertia load E_I.

5- CONCLUSION

Thus further improved, practical earthquake design of Reinforced Earth structures is probably still non-perfect and conservative. Nevertheless, it is based on a varied range of experimental and analytical results (very possibly unparalleled even in the field of conventional retaining structures) which are largely in agreement.

In conjunction with Reinforced Earth's inherent adaptability, then, this design approach provides a further guarantee of the safety of structures sited in seismically active regions.

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