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SEISMIC DESIGN FOR CENTRAL NUCLEAR EN EMBALSE - CORDOBA

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SUMMARY

Structures of Reactor Building and Service Building of the Central Nuclear en Embalse - Córdoba, were designed for seismic action performing static analysis and dynamic modal analysis, using finite element models. The main features of these models, and salient results of the analysis are reported in this communication.

1. INTRODUCTION

Argentine second Nuclear Power Plant, of 600 MWe, is at present in an advanced stage of construction near Embalse, Province of Córdoba.

It has a CANDU reactor, using natural uranium as fuel and heavy water as moderator and coolant. The reactor and the complete Nuclear Island of the Plant is supplied by AECL (Atomic Energy of Canada Limited).

The Plant is located in a moderately seismic zone, qualified as zone 1 in Argentine Earthquake Resistant Regulations (CONCAR 70). Due to building function, in design of structure were applied, in addition to above mentioned Regulations, requirements of AECB (Atomic Energy Control Board of Canada), which are indicated in relevant design specifications.

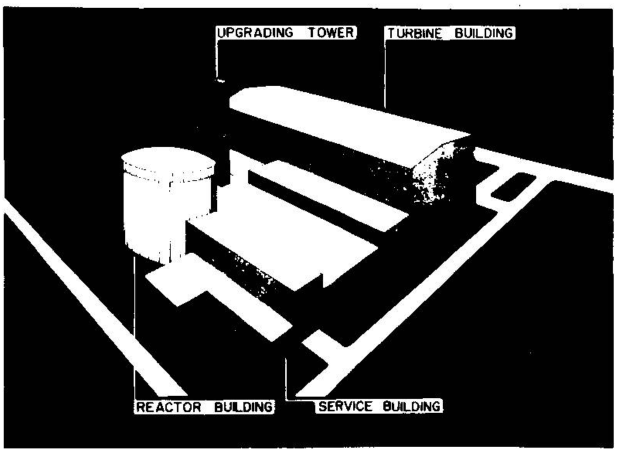

Fig. 1 - Aerial view of future Embalse Nuclear Power Plant

2. DESCRIPTION OF THE PLANT

Main buildings of the Plant include three adyacent buildings, of different type:

- Reactor building
- Service building
- Turbogenerator building

This communication refers only to the first two buildings, also called Nuclear Island, since same requirements are not applicable for Balance of Plant.

Reactor building includes an internal structure, in reinforced concrete, of several floors, which gives support and housing to Reactor and other equipment of primary system. Containment structure is a prestressed concrete cylinder with a dome of spherical sector shape, and has a protective function in case of a nuclear accident.

Service building includes spent fuel storage bays, heavy water upgrading tower, and a multistory building housing different systems for service of reactor. Its structure is in reinforced concrete.

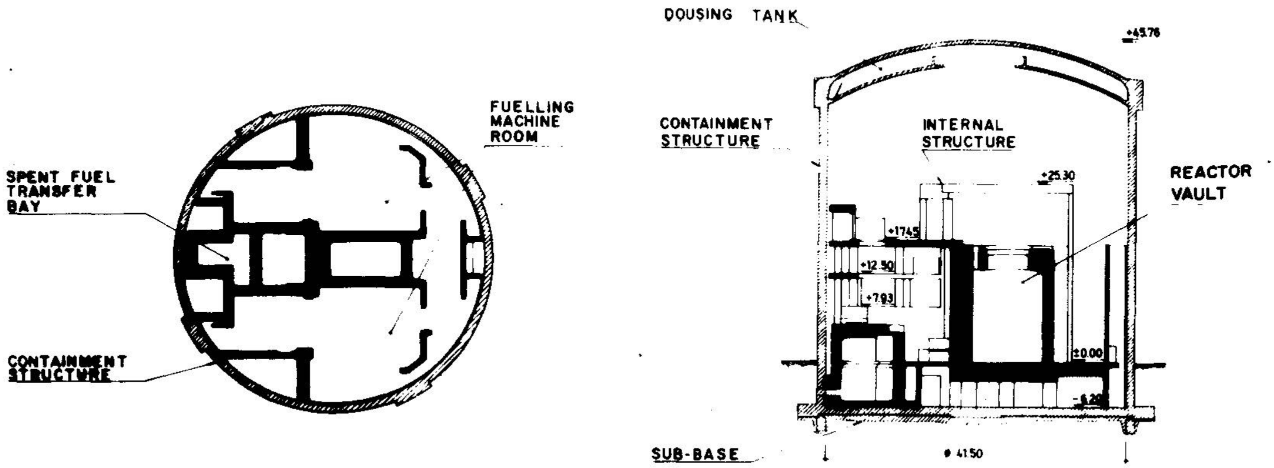

Fig. 2 - Reactor Building Plan and Elevation

3. DESIGN REQUIREMENTS

Basic design specifications, by AECL, define the "Design Basis Earthquake" (DBE) as the earthquake which has a probability of occurrence of one event per 1.000 years. Horizontal response spectrum for this earthquake is indicated in Fig. 3; to be used together with a vertical response spectrum, which is one half of horizontal response. Moreover, the "Site Design Earthquake" (SDE) is defined as the earthquake which has a probability of occurrence of 1 event per 100 years. Its spectrum is obtained from DBE spectrum, reducing its ordinates to one half.

AECL's design requirements, in general terms are:

Loading hypothesis to be considered:

I. Normal operation, with or without wind.
II. Normal operation with SDE earthquake.
III. Accident in primary sistem.
IV. DBE earthquake, with Plant shutdown.

For the first two hypothesis it is required that maximum stresses be below maximum normally allowable values. For the other two hypothesis (exceptional loading) increased values for maximum stresses are allowed.

For containment structure a fifth loading case, proof pressure, is specified; moreover, structure shall be leak proof, and this requires absence of cracks in every case, and absence of tensile stresses in the inner face, for normal loading hypothesis.

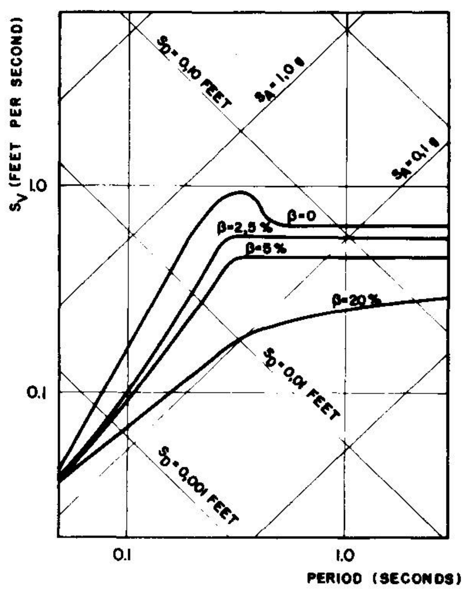

Fig.3 - Design Basis Earthquake Spectrum

To compute stresses, elastic behaviour of material is assumed.

4. EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

Taking into account different features of structures, in each case different procedures were used.

4.1 - Reactor Building Internal Structure

This structure was analyzed using a finite elements model and the earthquake effect was considered in a first approach by means of horizontal static forces.

These forces were obtained using accelerations previously computed by a dynamic analysis on a very simplified lumped mass stick model.

Later, a new dynamic analysis was performed, using another finite element model, somewhat simpler than the one used for static analysis.

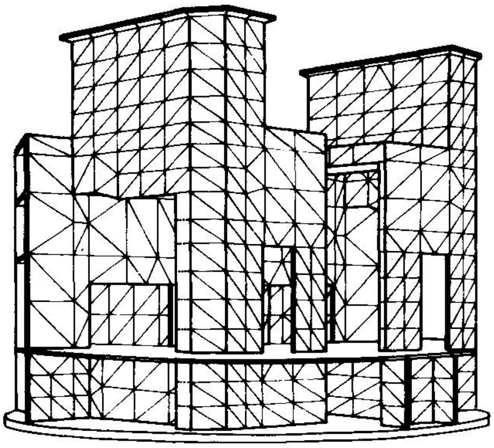

Fig.4 - Reactor Building Internal Structure - Finite Element Model for Static Analysis

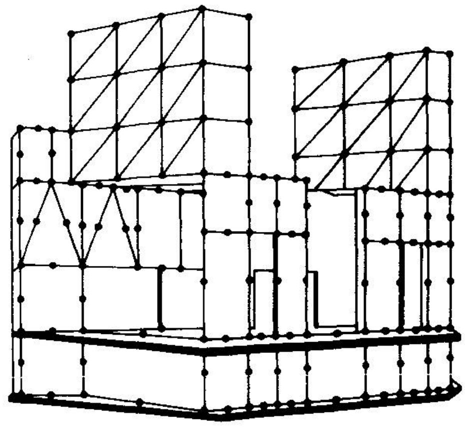

Fig.5 - Reactor Building Internal Structure - Finite Element Model for Dynamic Analysis

The figure does not show foundation soil, which is represented in the actual model by means of a cylinder of 20 m depth, and with a radius 20 m larger than base slab radius. Also containment structure mass is considered, in an approximate way, as a concentrated mass in the vertix of a piramidal membranal surface fixed to base slab contour.

This model has 173 degrees of freedom, and masses are concentrated in 36 joints.

Natural period of first mode is of 0.32 sec.

Design earthquake response was evaluated using a mean damping ratio of 4 % for the whole soil-structure system. By this value is intended to have a weighted mean value between a damping ratio of 2 % for concrete structure and 10 % for soil.

In dynamic response the first 12 modes are considered; modal composition was performed by square root of sum of squares.

Maximum acceleration of highest points of structure reaches 0.58 g.

4.2 - Reactor Building Containment Structure

For this structure earthquake effect is less important than other loading cases, like proof pressure or accident. Hence earthquake was not considered in static analysis, which on other hand was performed by means of an axisymetric model, since main loadings have this type of symetry.

Then dynamic analysis was performed using a finite element model which includes base slab, on elastic supports, perimeter wall, both domes, steel structures for dousing system, and water mass in dousing tank and piping.

This model has 116 degrees of freedom. First natural mode period is 0.42 sec. For design earthquake response damping ratio of 2 % was used for structure and 5 % for interaction with soil.

First 40 vibration modes were considered, performing modal composition by square root of sum of squares.

Overstresses in concrete due to earthquake are about 15 kg/cm2.

For elements of dousing system structure maximum horizontal accelerations of about 0.29 g were obtained.

4.3 - Service Building Structures

Due to extension and irregular shape of this building, its structure was divided into five sectors by means of expansion joints. Structures in general are formed by cast in place reinforced concrete columns, beams and slabs. Earthquake resistant elements are in general space frames, but there are also, for functional requirements, some concrete walls, which give to the structure more stiffness and strength. In some cases concrete walls were added to minimize mass excentricity with respect to stiffness center.

Some building sectors house only systems and equipment not directly related to Plant operation, e.g. maintenance shop, laboratories, etc. For these sectors it was considered sufficient a static analysis, i.e. compute natural period of the structure from deformation under horizontal static forces; determine seismic coefficient C for that period according DBE spectrum, compute total lateral force using this coefficient, and distribute it over the height, in accordance with CONCAR 70 Regulations. In this way were analyzed Sector II, whose natural period is 1.6 sec. in one direction and 1.1 sec. in the normal one, and C = 0.10 was assumed; and Sector III, whose natural periods range between 0.36 sec. and 0.75 sec., assuming C = 0.15.

For other sectors, housing equipments and systems of major importance, same procedure was used, but in addition a dynamic analysis was performed to compare its results with those of static analysis. These sectors are:

- Sector I, which houses spent fuel bays and main airlock for access to reactor building. For static analysis was used C = 0.10. Dynamic analysis showed a natural period of 1.2 sec, and accelerations of highest points of
- Sector IV, housing Control Room, Radwaste and D_2O tanks. Most of columns of this sector are fixed in basement slab, which rest directly on soil. Plan is almost square and structure is regular. Static analysis used C = 0.15. Dynamic analysis was performed with different hypothesis about stiffness of soil-structure system, including or not influence of masonry walls, which are not considered as earthquake resistant elements. Results comparison demonstrated that neither foundation soil flexibility nor walls modify substancially structure behaviour. Moreover, displacements and accelerations of each floor are sufficiently near to those of static analysis. Consequently these results were assumed valid, without any correction. Natural first mode periods range between 0.47 and 0.49 sec, and accelerations of highest points range between 0.296 g and 0.326 g in one direction, and 0.355 g and 0.368 g in the normal one.
- Sector V, which houses systems directly related with the reactor, like heavy water systems, and other important systems and equipments. From structural point of view, peculiarity of this sector are a very irregular plan layout, and a 70 m high very slender tower, with reinforced concrete walls, housing the heavy water upgrading tower. Static analysis used C = 0.15 for the base building and C = 0.10 for the tower. According to results previously obtained for Sector IV masonry walls and foundation flexibility were not accounted for, because walls distribution and foundation features are similar for both sectors. Instead of this, damping ratio influence was investigated, reanalyzing for β = 7%; 4% and 2%.

Comparison of results indicated that, even in the very conservative hypothesis of damping ratio of 2%, displacements and accelerations are much below those of static analysis. Specially for stresses verification in the tower it is necessary to reduce forces obtained by static analysis, even if, following recommendations of most Regulations, reduction was limited to 75 % of those values, while forces resulting from dynamic analysis are only 71 % of same values.

Natural first mode period is 1.63 sec. Accelerations of highest points of base building range between 0.285 g and 0.448 g in one direction and between 0.204 g and 0.347 g in the other, depending on assumed damping ratio. In the tower accelerations for highest points range similarly between 0.489 g and 0.968 g and variation of accelerations along the tower shows a minimum about the upper third of its free height, typical of this kind of building.

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STUDIES FOR COMPREHENSIVE ISO-RELIABLE SEISMIC DESIGN

by

R.Giannini,(1) A.Giuffrè,(2) P.E.Pinto(2)

INTRODUCTION

It is generally recognized that seismic resistant design must be based on a probabilistic treatment of the variables involved. In its simplest formulation, only the randomness of the input is considered, and the seismic action is specified by means of a single parameter (i.e. peak acceleration, velocity, etc.): the design is based on a selected fractile of this parameter. The next step involves modeling the ground motion by means of some kind of random process, thus introducing an additional source of variability on the response.

For a comprehensive approach, however, the uncertainties related to structure's behavior must also be accounted for. The latter may derive from a number of basic causes, including the scatter of material and element characteristics, as well as the uncertainty related to the analytical model.

In this study an attempt is made toward a comprehensive treatment of seismic reliability accounting for all the above mentioned uncertainties.

The level II method of reliability analysis is applied to assess the reliability of any design situation with respect to a predefined limit state involving some degree of structural damage. The results are presented in the form of charts, analogous to the familiar response spectra currently used for seismic design, and which give the design factor needed to garantee (with any chosen probability value) a prescribed level of non-linear response (max ductility ratio).

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Research carried out in the frame of the Geodynamics Project promoted by the National Research Council of Italy.

IDENTIFICATION OF THE RANDOM VARIABLES

A) SEISMIC INPUT

Several criteria have been proposed for evaluating the seismic risk at a site, given the historical and geological information relative to the surrounding region: see for ex. ref. |1|.

In the present approach, the seismic input is modeled by means of a suitable random process, scaled by a (random) parameter representative of the seismic intensity. In the numerical applications to follow, the peak ground acceleration has been choosen as the intensity parameter, and an extreme type I distribution has been adopted for the maxima of this parameter during a given reference period. Any other intensity parameter (ex.: peak ground velocity) or form of distribution could be introduced without difficulty.

The random process selected to simulate ground motion is a (unit intensity) non stationary gaussian process with a constant power spectral density: samples of such a process have been produced by means of the computer program: (PSQGN), |4|.

The generated samples have the following characteristics: duration 15 secs, central frequency ω=15,6 rad/sec, shaping function t_1=4 sec (duration of initial build-up), t_2=11 sec (end of stationary portion).

B) STRUCTURAL MODEL

For the purpose of the present parametric study, the bi-linear histeretic stiffness-degrading model proposed by Takeda |2| has been adopted. The normalized model is shown in fig. 1, where it is characterized by F_y, X_y, and K_2/K_1, which represent the yield force, the yield displacement, and the strain-hardening ratio, respectively.

The equilibrium equation in fig. 1 is expressed in terms of the ductility ratio $\xi=X/X_y$. The independent structural parameters which appear in the equation are: the undamped frequency $\omega=\frac{2\pi}{T}$, the damping ratio υ, the strain-hardening ratio K_2/K_1 (included in the restoring force $f(\xi)$), and the factor $\eta=F_y/A{\cdot}M$, where A is the peak ground acceleration and M is the mass of the model. (The latter is not an independent variable, being $M = =K_1/\omega^2$). The factor η will be referred to in the following as <u>design factor</u>, since it expresses the ratio between the (design) yield force F_y of the model, and the nominal peak inertia force: A·M.

The function a(t) at the right-hand side is the normalized (unit peak acceleration) random process.

Constant values υ=0,05 and K_2/K_1=0,05 have been used throughout the present numerical calculations, so that the structure's random parameters are represented by the natural period T and the yield force F_y.

ANALYSIS OF THE RESPONSE

The statistics of the response quantity ξ in eq. 1, given the values of the parameters: T and η , depend on the characteristics of the input process a(t).

A numerical simulation procedure, using 10 artificially generated accelerograms, has been carried out to determine the statistics of the response for a convenient range of the parameters T and η.

Mean value curves of the peak response (ductility ratio): μ, as a function of the period T and for various values of the design factor η are presented in fig. 2. As expected, the ductility demand decreases for the larger values of η, and for η>2,5 the (mean) structure's behavior never exceeds the elastic range.

In consideration of the small number of samples used in the simulation procedure, no attempt has been made to fit any particular form of distribution to peak displacement response. Based on previous investigations and on theoretical arguments |3|, |4| an extreme type 1 distribution has been assumed for this variable.

The number of the samples considered allows an estimate to be made of the coefficient of variation of μ: V_μ presented in fig. 3 as function of the period T and for various η.

For any given value of η, the regression line of V_μ as a function of T is nearly horizontal, showing no dependence in the mean of V_μ on T . By calculating the average of V_μ over all the periods T: $V_\mu av(T)$ for the various values of η the diagram in fig. 4 is obtained. fig. 4 shows that $V_\mu av(T)$ is nearly constant for all the η's, except for very small values (<0,3) for which the average observed dispersion is higher.

The C.O.V.'s of V_μ (not shown in fig. 4, calculated when averaging over T) are also approximately constant along the η's. From the forgoing analysis it can be concluded that V_μ can be taken as constant as far as the dependence on η is concerned, while the variability on T can be accounted for in the same way for all the η's.

The coefficient of variation finally adopted, also shown in fig. 4, corresponds approximately to the average plus one standard deviation of V_μ obtained averaging over the periods.

RELIABILITY MEASURE

A) LEVEL II PROCEDURE

The level II methods, as it is well known, yield safety indexes which, while avoiding complex convolution integrals, represent approximate measures of the probability of attainment of any explicity or implicitly formulated failure condition.

The method requires the definition of the failure boundary in the space of the basic variables, followed by a transformation of the variables from their original to a gaussian distribution.

The safety index β is defined as the minimum distance of the failure boundary from the origin, in the space of the transformed and normalized (zero mean, unit variance) variables.

In the present context, by 'failure' is meant the attainment of a selected level of ductility response: μ. The index β gives then the probability that the response will not exceed such level.

For illustration, a simplified case involving only two random variables is presented in fig. 5.

A particular design situation is considered, with T=0,6 sec and η= =1,5 , so that the r.v. present are the peak ground acceleration A and the structure's response μ. The boundary is defined by the condition: μ=3, and the minimum distance is found to be: β=1,86 (to which corresponds a probability of 3,14 10^{-2}). The coordinates of the 'checking point' measure the probability of the two r.v. considered when the limit state is attained in its most probable point.

The level II safety analyses have been performed by means of the computer program described in |2|. The program can deal with any explicitly or implicitly defined failure boundary (g-function).

At each step of the searching procedure for the minimum distance the original g-function (and its derivatives) is modified, as a consequence of the transformation of the original basic r.v. into normal ones.

B) RESULTS

The direct results of the safety check analyses are of the form illustrated in figs. 6 and 7. The curves in these figures give the index β as a function of the mean natural period $\bar{T}$, for various values of the mean design factor $\bar{\eta}$; all cases refer to a failure condition defined by $\mu=3$.

In fig. 6 both the seismic input (the peak acceleration A plus the random process) and the structural parameters (F_y and T) are considered as random, with the indicated types of distribution and values of the coefficients of variation.

Fig. 7 compares some of the curves in fig. 6 with the corresponding ones obtained for the case of deterministic structural behavior ($V_{F_y}=V_T=0$). It is seen that consideration of the randomness of the structure's behavior reduces the overall reliability increasingly with increasing $\bar{\eta}$ and natural period $\bar{T}$.

With the particular values of the statistical parameters adopted ($V_{F_y}=0,15$, $V_T=0,20$), which are not unrealistic, the effect can be significant even in the normal range of designs.

For instance, for $\bar{T}=1,0$ sec and $\bar{\eta}=1,5$, the value of β is reduced almost by a half point, which corresponds roughly to a reduction of one order of magnitude on P_S.

All the curves in figs. 6 and 7 steadily increase with increasing $\bar{T}$, thus indicating greater reliability for longer period structures. This is partly due to the assumed frequency content of the artificial accelerograms, which were choosen to represent hard soil conditions, and thus did not include significant power in the low frequency range.

The final presentation of the above results is obtained rearranging the diagrams in fig. 6 by drawing horizontal sections through them. Each line collects the $\bar{\eta}$ values versus period $\bar{T}$ to which corresponds a constant level of reliability. In fig. 8 the new curves are reported: they furnish the mean design factor, i.e. the ratio between the mean yield force to the mean peak ground acceleration, required in order that structure's response does'nt exceed (with a selected degree of reliability) a specified level of ductility, as a function of the mean natural period of the structure.

CONCLUSIONS

A procedure has been illustrated for carrying out level II safety checks of non-linear random s.d.o.f. structures under random dynamic seismic excitation. The procedure is general with respect to the structural model actually used and to the probabilistic definition of the variables. Some results are presented, with comments on the influence of the randomness of the structure's behavior on the overall reliability.

Finally, the procedure has been employed to construct iso-reliable design response spectra. Each spectrum corresponds to a specified level of reliability against the exceedance of a limit-state defined in terms of maximum ductility response. With different sets of spectra available for different sets of statistical parameters and allowable ductilities, their use for design will be straightforward and general. The required input consists on the calculated mean natural period of the structure, and on the mean maximum peak ground acceleration during a reference period (ex. fifty years) relative to the site of interest. The output is the average strength the structure must be designed with.

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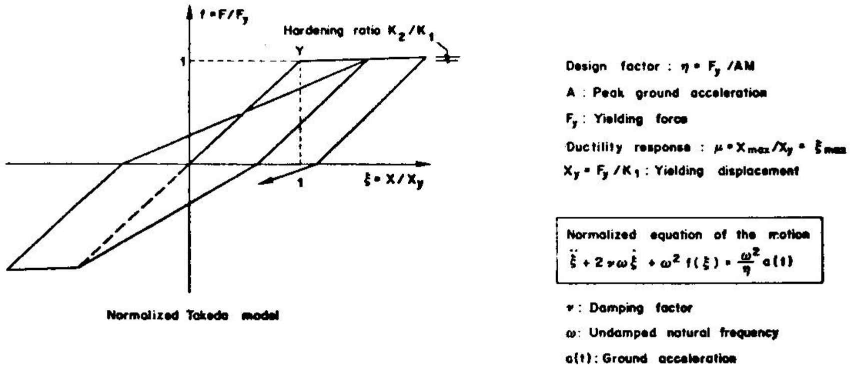

f = F/F_y
Hardening ratio K_2/K_1
Y
1
1
ξ = X/X_y
Normalized Takeda model
Design factor : η = F_y /AM
A : Peak ground acceleration
F_y : Yielding force
Ductility response : μ = X_max/X_y = ξ_max
X_y = F_y / K_1 : Yielding displacement
Normalized equation of the motion
ξ̈ + 2νωξ̇ + ω² f(ξ) = (ω²/η) a(t)
ν : Damping factor
ω: Undamped natural frequency
a(t): Ground acceleration

Fig. 1

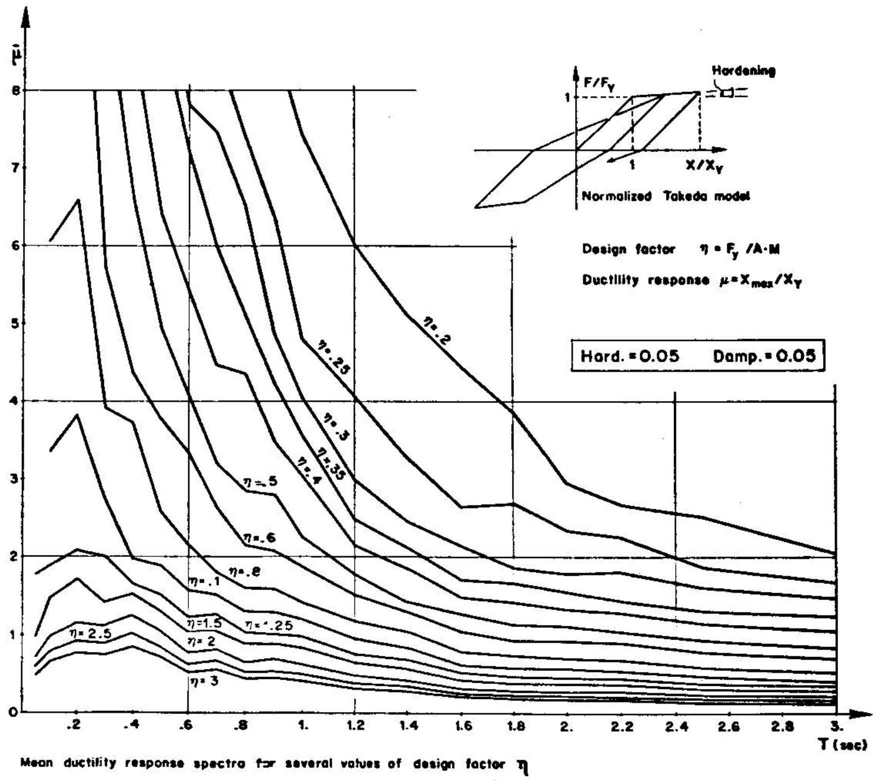
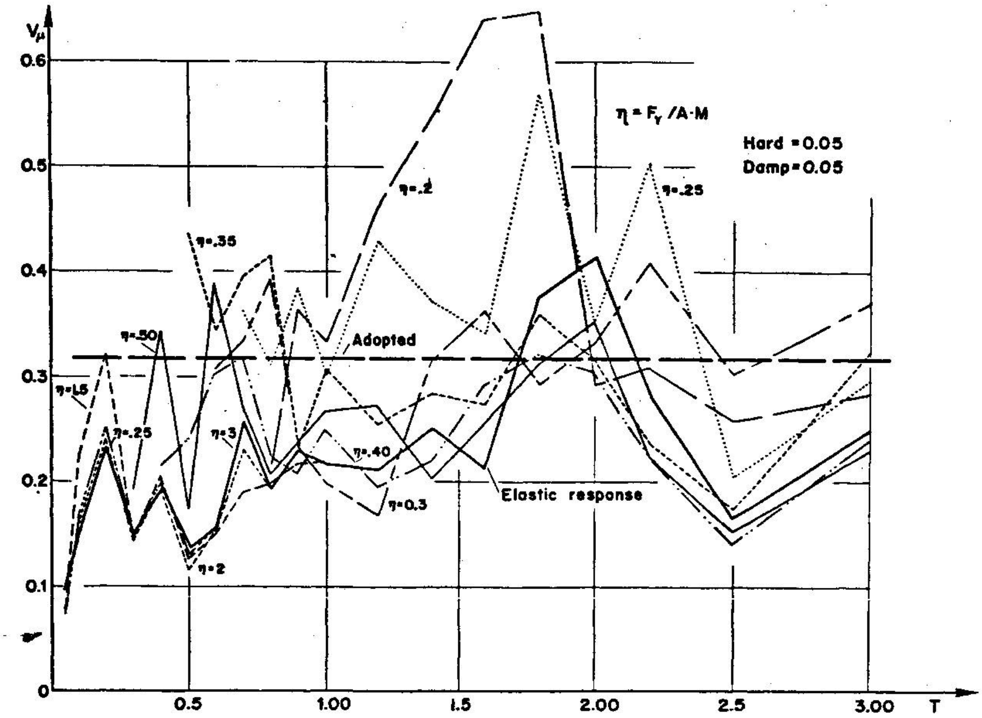
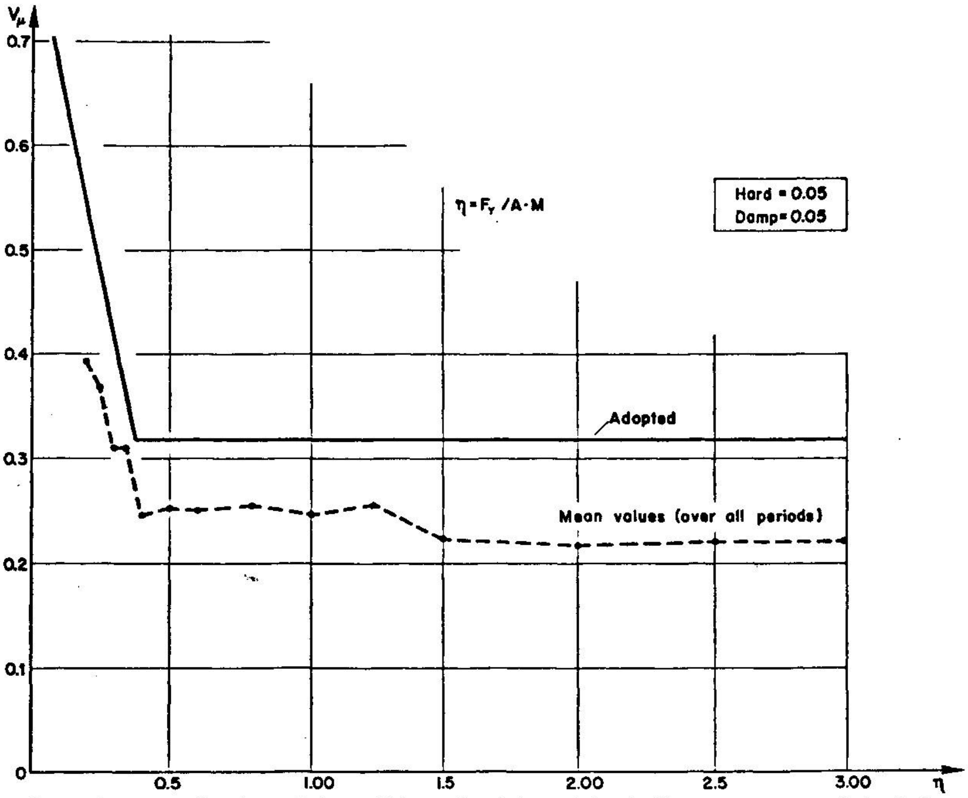

μ̄
F/F_Y
Hardening
X/X_Y
Normalized Takeda model
Design factor η = F_y /A·M
Ductility response μ=X_max/X_Y
Hard.=0.05 Damp.=0.05
η=.2
η=.25
η=.3
η=.35
η=.4
η=.5
η=.6
η=.8
η=1
η=1.5
η=1.25
η=2
η=2.5
η=3
T (sec)
Mean ductility response spectra for several values of design factor η

Fig. 2



Coefficients of variation of the ductility response spectra for various values of the design factor

Fig. 3



Mean values over all periods of the coefficients of variations of the ductility response versus design factor η

Fig. 4

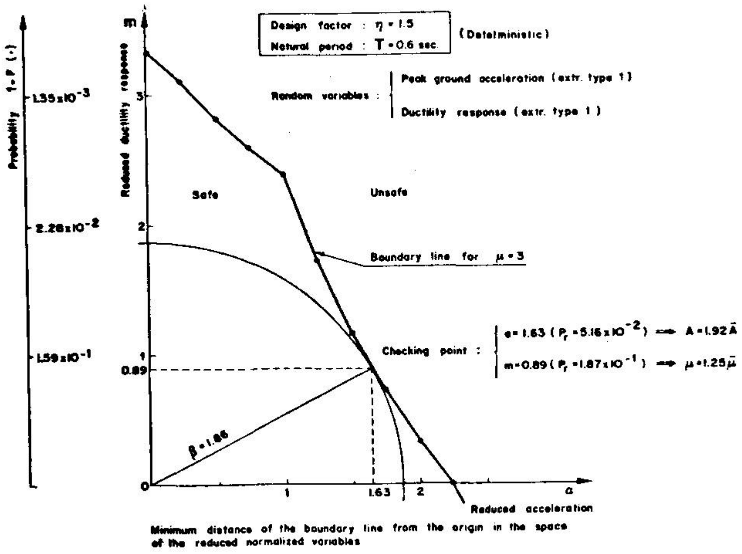
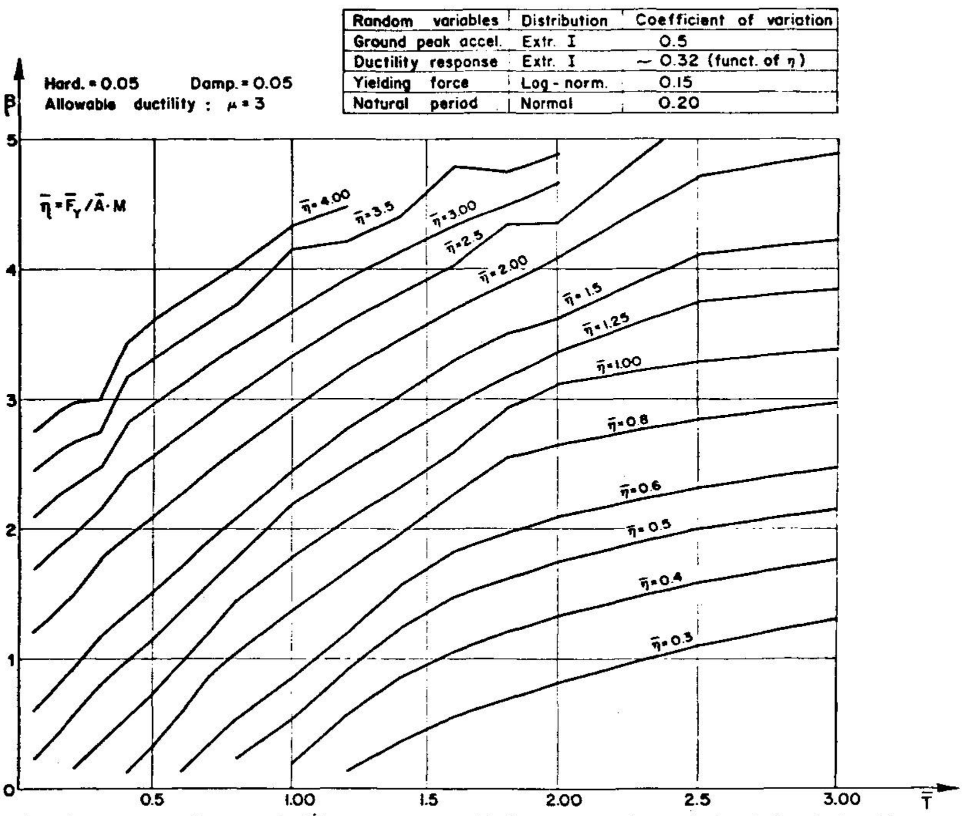
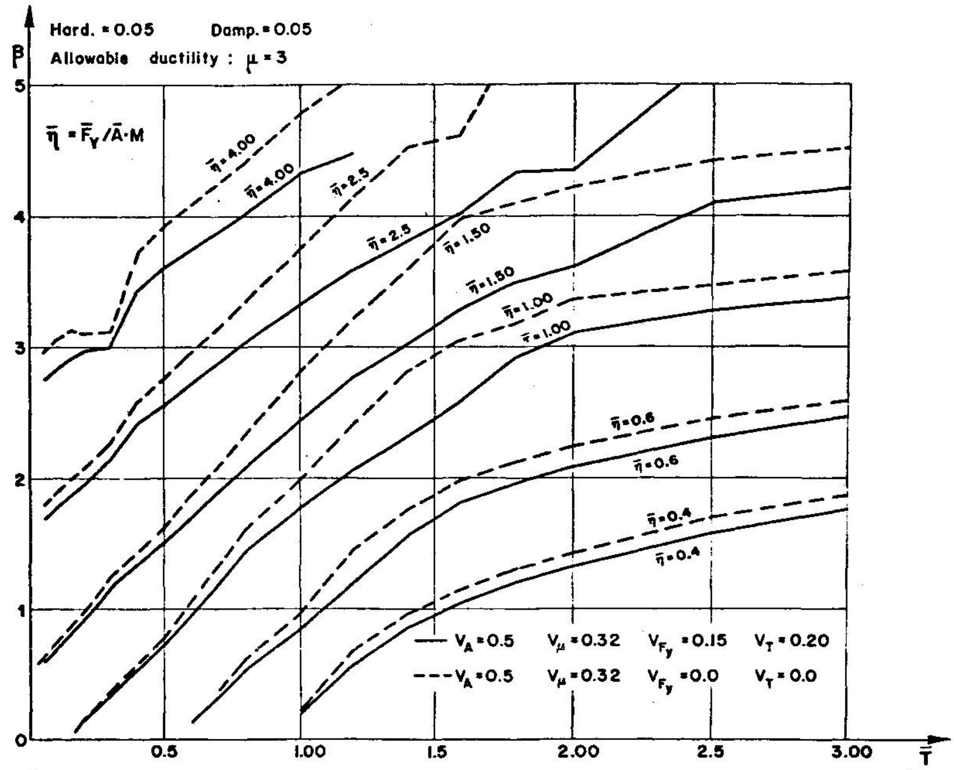


Fig. 5



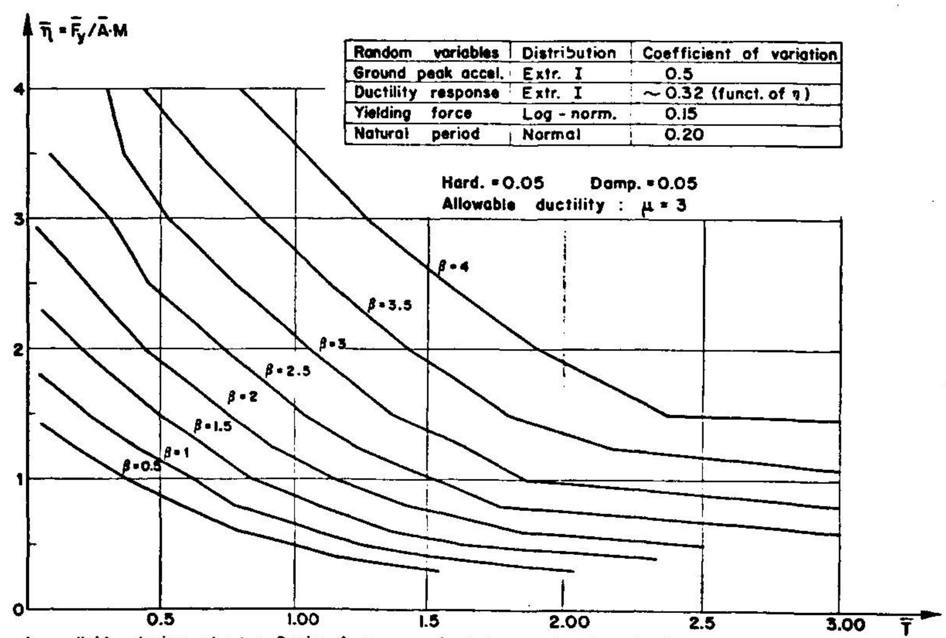
β values corresponding to ductility response $\mu = 3$ for several values of the design factor $\bar{\eta}$

Fig. 6



β values corresponding to ductility response μ = 3 for several values of the design factor $\bar{\eta}$

Fig. 7



Iso-reliable design chart : Design factor required to attain the ductility response μ = 3 for several reliability levels

Fig. 8

SOIL CONDITION EFFECTS ON SEISMIC STRUCTURE DESIGN

L'EFFET DE LA CONDITION DU SOL SUR LE PROJET SISMIQUE DES OUVRAGES

WIRKUNG DES BODENBESTANDS AUF DER SISMISCHE ANSCHLAG DEN BAUWERKEN

by Michele Maugeri, Ass.Professor of Soil Mechanics University of Catania - Faculty of Engineering.

Abstract

The influence of soil condition on the behaviour of buildings is now being considered in Italy after the Friuli Earthquake by means of extensive area microzonation. .

But sometimes microzonation methods cannot be applied successfully and a direct evaluation of the interaction of soil layer characteristics with structural features cannot be avoided.

In this paper the effect of two different soil layers of variable depth is evaluated on eighteen different structure patterns.

Lastly the reduction of the seismic shaking by means of a deep pile foundation is quoted.

Resumé

En Italie, après le tremblement de terre en Friaul, on est en train de considérer l'influence de la condition du sol sur les bâtiments par une vaste microzonisation des zones.

Pourtant il arrive quelquefois que les methodes de microzonisation ne peuvent pas être appliquées aved succès, donc il est impossible d'éviter une directe évaluation de l'intéraction des caractéristiques des couches de sol avec les configurations structurales.

Ici on évalue l'effet de deux differentes couches de sol, ayantes une profondeur variable, sur dix-huit différents modèles d'ouvrages.

Enfin on cite la réduction des vibrations sismiques par des fondations profondes à piliers.

Zusammenfass

Die Einfluss des Bodenbestands auf dem Bauwerkshalten wird jetzt in Italien, nach dem Friaulischen Erdbebens, in Betracht gezogen, vermittelst einen ausgedehnten der Bodenfläche.

Manchmals, aber, ist es nicht möglich die Methoden mit Erflog anzuwenden, deswegen ist es nötig eine direkte Auswertung des Wechselwirkungs des Kennzichnungs des Bodenbestands mit den Besonderheiten des Bauwerken anzuwenden.

Hier wird die Wirkung von zwei veschiedenen Bodenschichten mit veränderlicher Tiefe an achtzehn vershiedenen Bauwerkmustern ausgewertet.

Zuletzt die Reduktion von der seismische Eigenschwingung vermittelst ein tiefes Stoßfundament wird angeführt.

1. INTRODUCTION

The usual way of evaluating seismic forces when designing structures is to multiply the weight of the construction by a series of coefficients (Tezcan 1972). Two of these take the soil effect into account.

- the reduction factor, reflecting the effect of epicentral distances in relatively soft soils;
- the amplification factor due to soil layers and their interaction with the structure.

In many countries, particularly Japan (Ohsaki, 1969, 1972; Kobayashi and Kagami, 1972), this way of approaching the question has led to the microzonation of extensive areas. Italian researchers are also doing the same for the districts hit by the recent earthquake in Friuli (Giorgetti, 1977).

However microzonation methods are not fully successful when the local soil conditions present one of the following characteristics :

- vicinity to hills which disturb the soil amplification response (a very frequent case in Italy);
- presence of deep foundations of various types and depths;
- vicinity of the (largest or mean) period of microtremors to the predominant period of the soil layer and/or vicinity of the predominant period of the soil layer to the natural periods of the structure;
- vicinity to the epicenter of the examined zone.

In the last case the soil amplification technique (Idriss and Seed, 1969; Hayes et all., 1971) which sometimes gives an amplification coefficient of up to 20 - 25 for velocity (Kobayashi, 1977; Kobayashi and Nagahashi, 1975) cannot be successfully applied to plasticization and/or soil rupture in the epicentral areas. A great impulse in the direction of measuring slip vectors in the field during earthquakes (Papastamatiou, 1976) is that many collapses are due to unacceptable structure deformation. The influence of the microtremor period and of the predominant period of soil layer is so remarkable that different methods of microzonation, as shown in fig. 1, are based on the largest and the mean microtremor period (fig. 1a) the largest amplitude and predominant microtremor period (fig. 1b) and on the predominant period for each zone (fig. 1c).

During the recent Friuli Earthquake, for a series of caisson piers of variable height which will allow the motorway to cross the Cavazzo Lake, the velocity spectrum was near the oscillation periods and some piers were severely damaged at their bases (Bo et all., 1976).

2. SOIL STRUCTURE INTERACTION

The complex interaction of soil layer characteristics and its consequent predominant period with structural features, particularly, rigidity and ductility, can be carefully predicted if the design spectrum for a given structure resting in a given soil layer is known. This process of integral design for each case cannot be avoided for very large strctures or for simple structures with a high degree of repetitivity both on the structural pattern and in the condition of the subsoil.

In fig. 2 three typical structures, the last of which (diagram C) is very commonly used for residences in Friuli, are taken into consideration. The design spectrum is visible in fig. 3 in terms of maximum acceleration, in fig. 4, in terms of velocity and in fig. 5 in terms of displacement. The spectra shown are those of the May 6th Friuli Earthquake at Tolmezzo, as representing rock substratum, and at Codroipo as typical for soft ground, both with a damping coefficient of 10% (CNEN - ENEL, 1976). The displacement effect, severer at Codroipo than at Tolmezzo, is evident.

In fig. 6 the influence of rigidity is plotted, and in fig. 6a on the natural periods of the structures, fig. 6b the maximum acceleration and fig. 6.5 the displacement from which it is derived.

At this stage, differences in rigidity are due to six different pier dimensions (fig. 2d) and different pillar reinforcement percentage making a total of eighteen (18) different patterns. It is obvious that each pattern also has different ductility characteristics.

From the point of view of soil characteristics for different layer depths and different shear wave velocities, a particular predominant period, quoted in fig. 7a (Maugeri, 1976), corresponds and consequently various maximum ground accelerations (fig. 7b).

When the predominant period of soil layers of different depths reaches the field of the natural period of structures, the maximum accelerations are amplified as shown in fig. 8 for a shear wave velocity of 36 m/sec, corresponding to very soft soil, and in fig. 9, for a shear wave velocity of 480 m/sec, corresponding to stiff soil. It is clear from the graphs that stiff soil amplifies accelerations more than soft soil (though the latter demagnifies accelerations in the epicentral areas). The seismic stability of pattern C (fig. 2c) 9.80m high and 4.00m wide with 40 x 30 cm pillars and a 4.02 cm^2 reinforcement is not reached in the elastic field, but requires plasticisation of the section with a ductility factor of 2.7 in the case of structures resting in a rock and of 3.2 in the case of structures based on 10m of stiff soil. The ductility required was less than the 5.5 offered by the structure alone in the ultimate state, which can usually be reached in r.c. structures of this type (Benedetti and Vitiello, 1976).

3. CONCLUSION

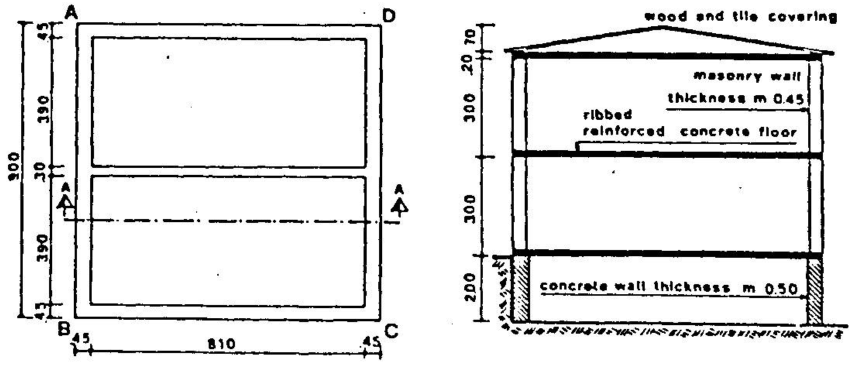
From the preceeding section the strong influence of geotechnical soil characteristics is clearly evident on the structural safety requirements, particularly in terms of strength, deformability and then ductility.

When the predominant period of the soil layer is near the natural period of the building considered, the soil structure interaction reaches its maximum effect consisting in a magnification of acceleration and/or displacement which affects the structure (Maugeri, 1971; Bo et all. ; 1978). When the predominant soil period is different from the period of structures the interaction effect diminishes and can be negative, as results from fig. 8 (except in the case of a soil layer depth of 5m. when a small acceleration magnification still remains).

On the other hand, when the subsoil is formed of rock and the predominant earthquake period is very different from the period of structures the soil structure interaction effect is practically absent.

This is the case of the house of the keeper of the Ambiesta dam (fig. 10), a few metres away from where the maximum acceleration value was recorded (Tolmezzo station) during the Friuli earthquake.

The natural period of the construction resting on calcareous rock (Martinis, 1977) and made of both concrete and the bearing masonry was 0.023 sec. (taking into account the reduction in the stiffness due to the presence of the windows in the masonry).



a) PLAN

b) SECTION A-A

Fig. 10: Structural sheme of the house near the Ambiesta dam.

As a result, there was no appreciable amplification in the structure and it was virtually undamaged (small signs of damage which appeared during the second shock on 15th September 1976 could have been favoured by the increase in the original natural period due to the plasticisation during the main shock of 6th May 1976).

Lastly the relevance of a deep embedment as a measure to reduce the seismic shaking must be quoted.

In the case of the new terminal at Catania Airport, which will be built in a so far non seismic area above a clay soil deposit described by Maugeri (1977), when the response spectra of Codroipo (see figs. 3, 4, 5) is imposed, the seismic effect was reduced by 16% of that of a surface foundation, by taking the embedment due to the pile foundation depth of 40m into account.

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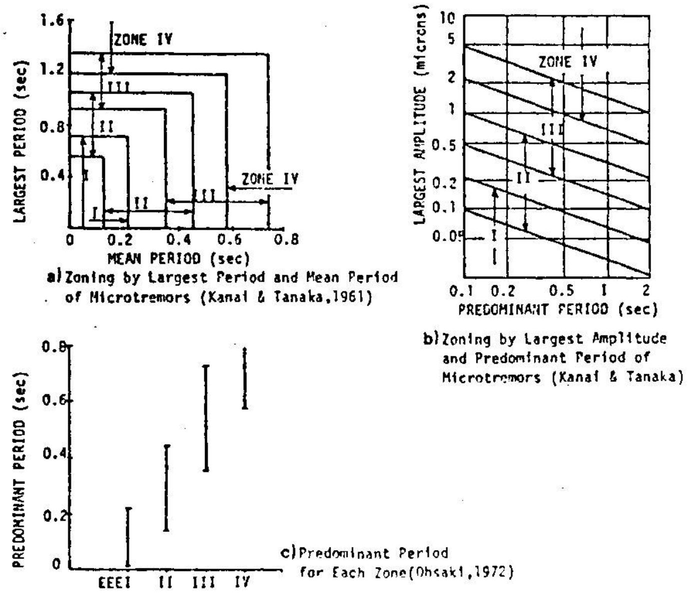

Fig. 1: Dynamic interpretation of Japanese zoning adopted by Ministry of constrution since 1951.

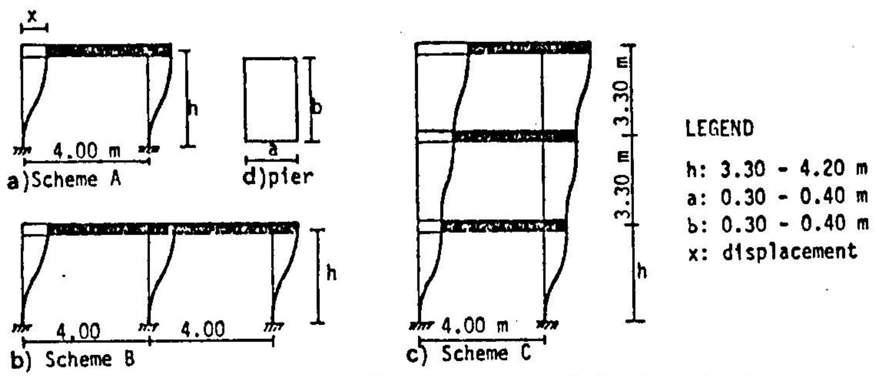

Fig. 2: Strctures examined with several height and pier dimensions.

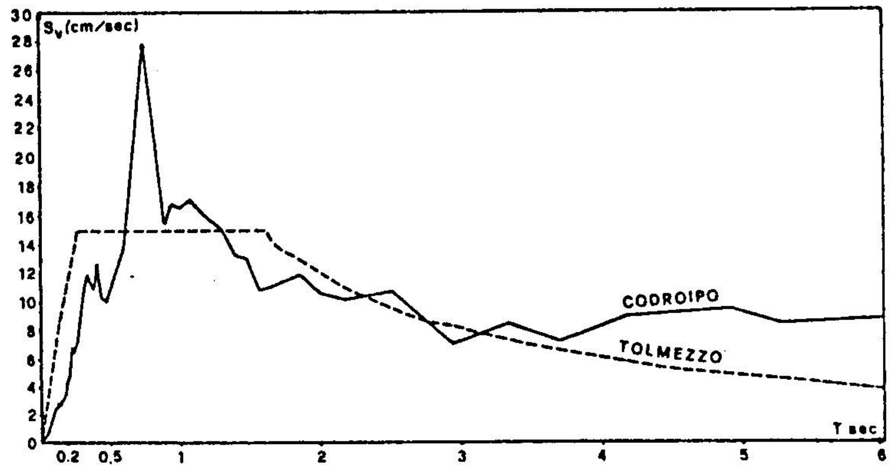

Fig. 4: Design velocity spectrum; Friuli Earthquake 1976.

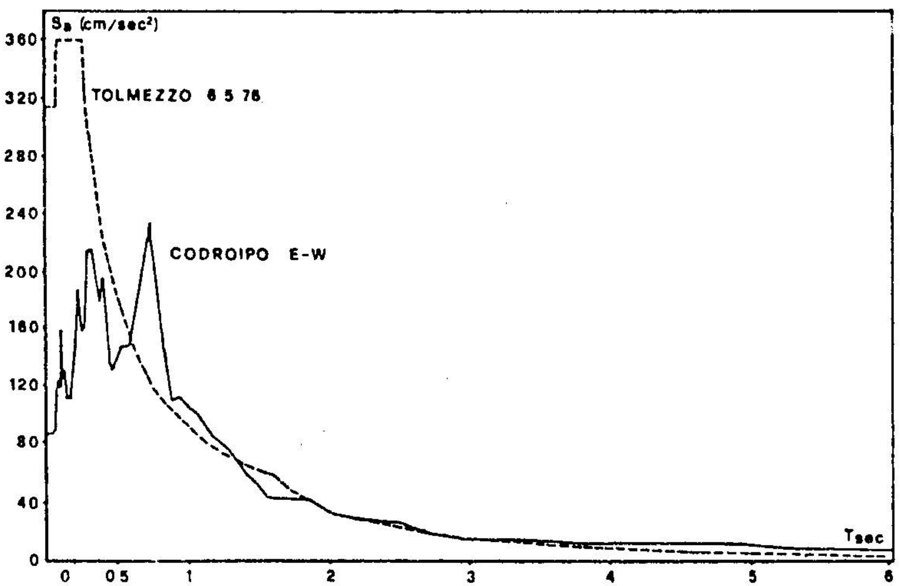

Fig. 3: Design acceleration spectrum; Friuli Earthquake 1976.

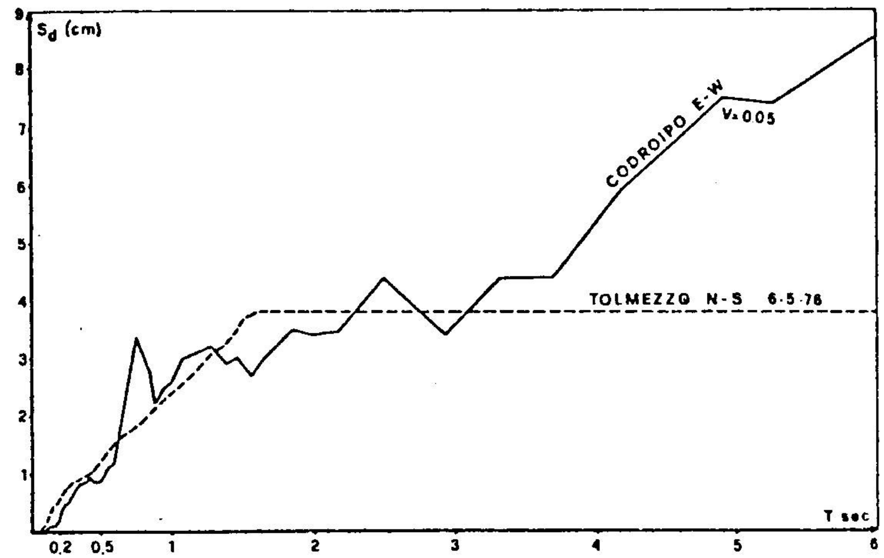

Fig. 5: Design displacement spectrum; Friuli Earthquake 1976.

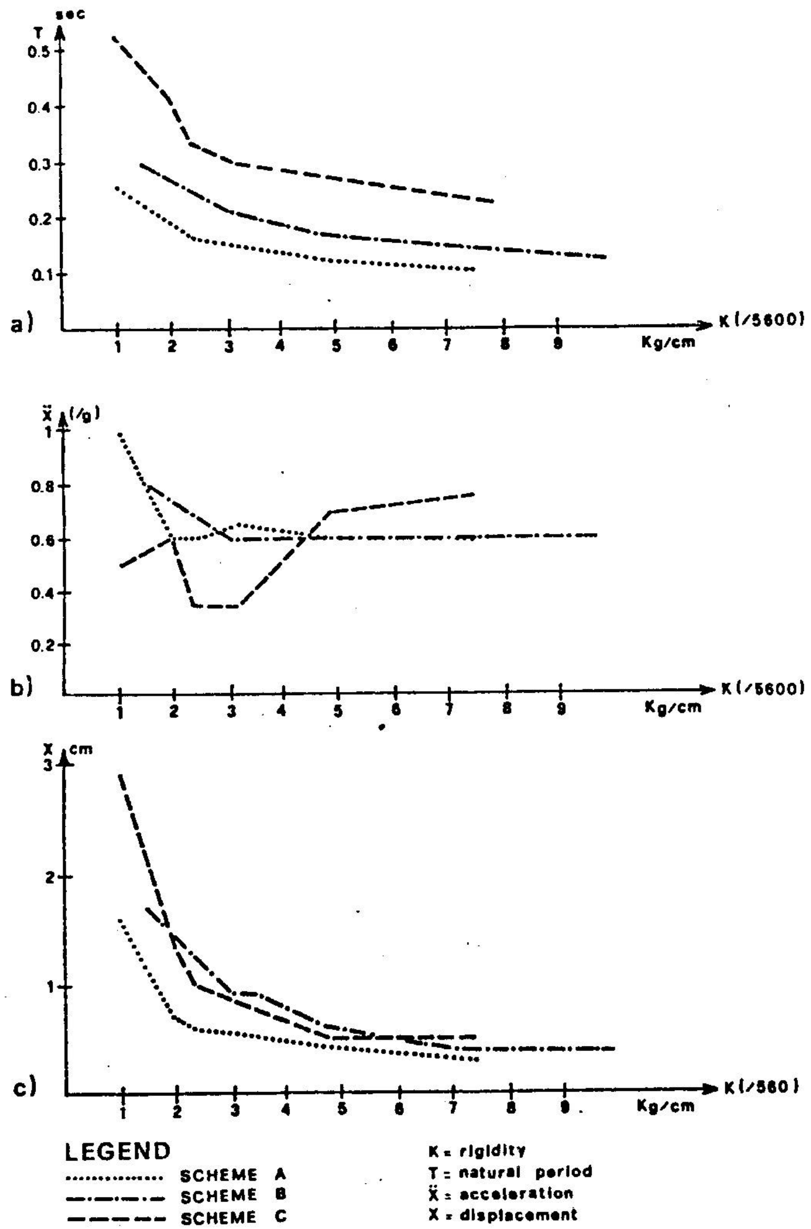

Fig. 6: Influence of rigidity on the natural periods, max structure accelerations and displacements.

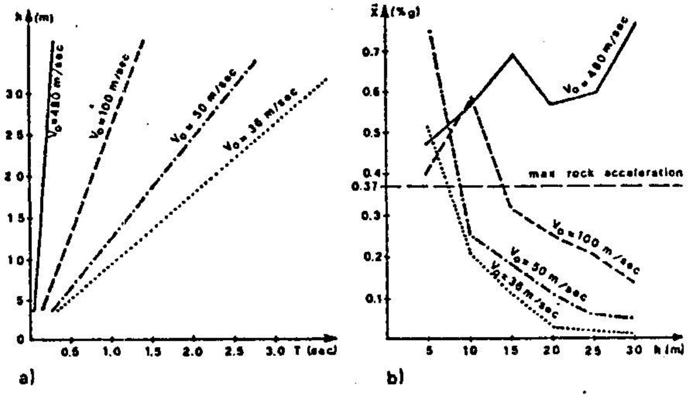

Fig. 7: Influence of the depth of the soil loyer on its predominant period and on the max ground acceleration for a given sheor wave velocity.

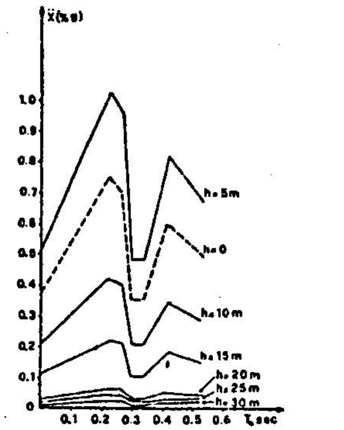

Fig. 8: Influence of structural natural period on its max acceleration resting on soft soil loyer of different depth.

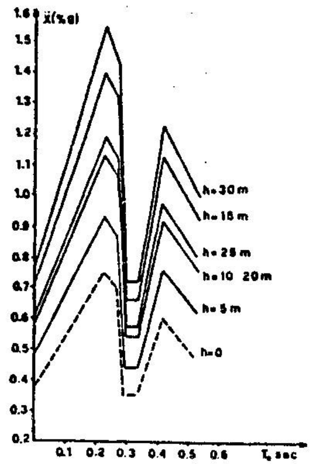

Fig. 9: Influence of structural natural period on its max acceleration resting on a stiff soil loyer of different depth.