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SEISMIC EARTH THRUST AGAINST MASSIVE BUILDING WALLS

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SUMMARY

The problem of earth thrust for a massive building in a seismic environment is analysed. Inertial and kinematic soil–structure interactions are considered by modelling the soil and the structure together. The problem is solved in the frequency domain by using the computer code FLUSH. Results show that the horizontal component of the seismic earth thrust is much greater than that obtained when applying the Mononobe–Okabe theory. This study establishes a set of conclusions and recommendations for both design and practical purposes, and proposes future lines of investigation.

INTRODUCTION

In the engineering and design of embedded massive buildings in nuclear power plants, some problems appear to be related to the evaluation of seismic earth thrust. In fact, the consideration of the seismic excitation introduces a set of forces, in addition to the static ones, often difficult to evaluate on account of the intrinsic uncertainty of the seismic excitation and the randomness of the process (type of seismic wave, direction of propagation, etc.).

Traditionally, seismic earth thrust has been evaluated on the basis of the earliest study of the problem developed by Mononobe and Matsuo,[1] and Okabe,[2] in the 1920s. Known as the MO method, it is based on a modification of the Coulomb theory of static earth pressure, introducing static equivalents of horizontal and vertical inertial forces. It assumes that a soil wedge behind the wall is at the point of incipient failure, and that the maximum soil shear strength is mobilized along a potential sliding plane surface which passes through the wall toe. Thus, the total earth thrust—static plus dynamic—is evaluated from the sliding surface that produces the minimum active thrust. The equivalent forces are computed by multiplying the wedge mass by the horizontal and vertical earthquake accelerations. This last aspect introduces an important requirement: the soil wedge behaves as a rigid body. Also, the inertial wall forces are neglected, and thus soil–structure interation effects are not taken into account. The soil pressure distribution law may be considered as triangular,[3] with maximum soil pressure at the surface, decreasing uniformly to about zero at the bottom of the wall, when both the structure and the soil are supported on bedrock. This leads to a resulting seismic earth thrust acting at $\frac{2}{3}H$, H being the wall height, from the base of the wall. The MO method became a very popular approach to computing the seismic earth thrust problem, due to its inherent simplicity, but the above-mentioned assumptions greatly limit its application, so it is now seldom used in the design of walls of massive buildings such as those for nuclear power plant. Soil–structure interaction phenomena can become important and they must be taken into account.

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Another point is that the MO method does not consider seismic soil amplification, which is related to factors such as the wall height, nonlinear soil behaviour and the different wave types involved in seismic excitation. A detailed discussion of these points is found in References 3 and 4. The influence of surface waves (Rayleigh waves, essentially) on seismic earth thrust has been pointed out by Gomez-Masso *et al.*[5] and by Navarro and Samartín.[6]

The main object of this paper is to analyse the problem by a numerical method. Typical nuclear power plant buildings (reactor, polishing and electrical buildings), founded on different soil types, are considered in order to evaluate the influence of soil–structure interaction effects on seismic earth thrust.

ANALYSIS OF THE PROBLEM

Cases studied and numerical method

Two basic structure–soil systems are considered. The first, called problem 1 and shown in Figure 1(a), belongs to a building founded on a rock half-space and surrounded by a well-compacted granular backfill. This could be a reactor building in a nuclear power plant. In this type of facility the reactor building is embedded in the soil and is founded on competent rock. The second system, called problem 2 and shown in Figure 1(b), represents a building founded on a mixed system formed by granular backfill on very competent hard rock, which in turn stands on a perfectly rigid half-space. This greatly reduces the vertical dimension of the model in comparison with the previous case. The main difference between the two models is in their horizontal structural responses, as a result of the different soil rigidities. In fact, the soil–structure interaction may become more important in Problem 2 than in Problem 1.

Current trends in the analysis of dynamic soil–structure interaction tend to solve the governing equations of motion in the time domain in order to account for nonlinear and plastic soil behaviours in a more realistic manner. However, the problem presented here is analysed in the frequency domain. Thus the nonlinear soil behaviour is considered in only an approximate way, and computational time may be saved. The two cases have been analysed by the finite element method using the well-known soil–structure interaction program FLUSH.[7] With this program the structure and the soil media can be modelled. Although two-dimensional (2D) situations only can be considered with this code, three-dimensional (3D) effects may be taken into account by using viscous boundaries acting along the planar surface of a slice of soil on which the structure is located. A detailed description of this last aspect may be found in Reference 7. In order to account for 3D effects, some previous dynamic analyses of the structures considered in this paper were carried out by comparing responses obtained from FLUSH analyses with those calculated from response spectrum analyses of lumped mass structural models in which the soil–structure interaction was considered by means of a set of soil spring and damping coefficients. In this way, the input parameters of FLUSH that govern 3D simulation were chosen. The soil behaviour is considered to be viscoelastic and nonlinear, and the program solves the governing equations of motion accounting for this last effect in approximate form. The seismic excitation is obtained by imposing, at the base of the model, a predetermined motion obtained from the surface motions by means of a deconvolution process.

The approximations mentioned above and the limitations of the FLUSH program, should be borne in mind when interpreting the results presented in this paper.

Soil properties

As already mentioned, the behaviour of the soil is considered to be viscoelastic. The dependence of the soil's dynamic properties (shear modulus and damping ratio) on shear strain is evaluated

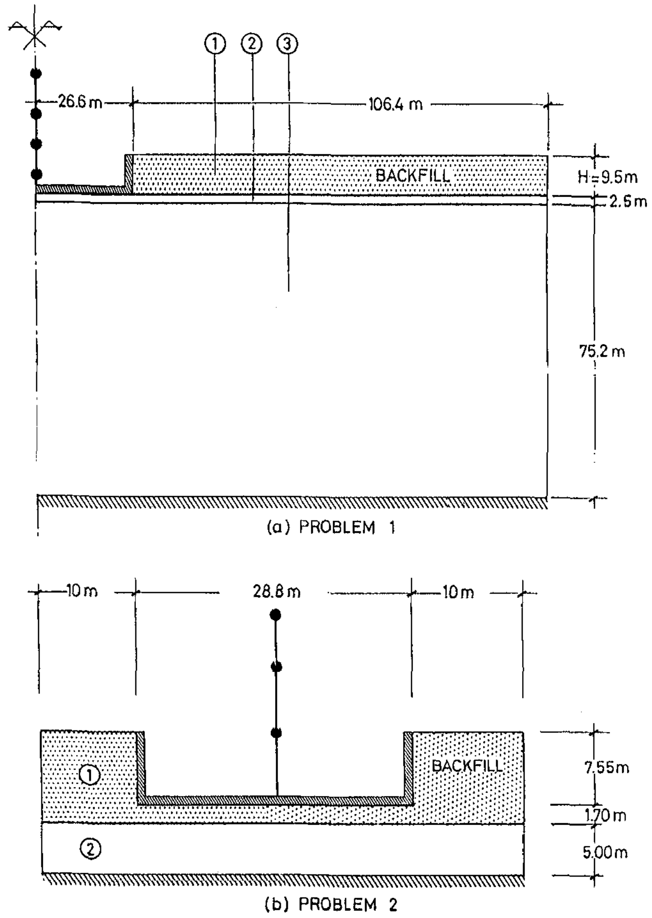


Figure 1. Soil–structure systems

from the model proposed by Hardin and Drnevich[8] for granular materials. The behaviour of soft and hard rocks is assumed to be elastic and linear. The different soil properties (velocity of shear wave, β; dynamic Poisson ratio, ν; and specific weight, γ) are summarized in Tables I and II for the two problems under consideration.

Table I. Dynamic soil properties for Problem 1

Case	Layer	Material	β(m/s)	ν	γ(t/m^3)
	1	Granular backfill	300	0·4	2·0
1. Very hard rock	2	Hard rock	2500	0·18	2·7
	3	Hard rock	2500	0·18	2·7
	1	Granular backfill	300	0·4	2·0
2. Intermediate rock	2	Transition rock	650	0·3	2·4
	3	Sedimentary rock	1200	0·3	2·4
	1	Granular backfill	300	0·4	2·0
3. Very soft rock	2	Softrock	650	0·3	2·3
	3	Softrock	650	0·3	2·3

Table II. Dynamic soil properties for Problem 2

Case	Layer	Material	β(m/s)	ν	γ(t/m^3)
	1	Granular Backfill	300	0·4	2·0
1. Fractured hard rock	2	Fractured rock	1800	0·18	2·7

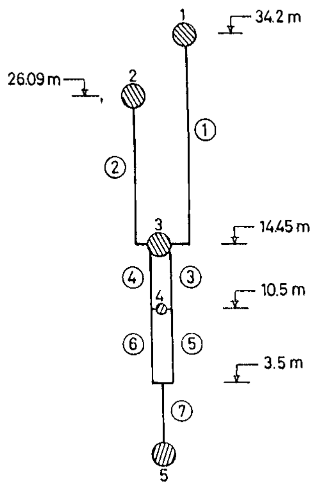
For Problem 1 (Figure 1(a)), three cases are considered. In case 1 a backfill layer rests on a half-space of greywacke. In case 2 the half-space represents a sedimentary rock with a transition layer of altered rock near the surface. In case 3 the half-space is formed by a very soft sedimentary rock.

For Problem 2 (Figure 1(b)), the building is founded directly on backfill. The backfill rests on a layer of altered greywacke, which is placed on an infinitely rigid half-space. This means that radiation damping decreases very much in comparison with the previous cases, and thus structural responses increase.

Structural models

Three different buildings were considered in the analyses. The most massive is a reactor building (120,900 t); the second is a typical electrical building (24,100 t), and the third a polishing building (27,400 t). All are actual buildings in existing Spanish nuclear power plants. For these buildings simplified models are obtained from more complicated structural models, and they show equivalent dynamic responses in the fundamental modes of vibration. This allows simplification fo the problem, and reduces computer time.

The models used for the analyses are shown in Figure 2 (reactor building), Figure 3 (electrical



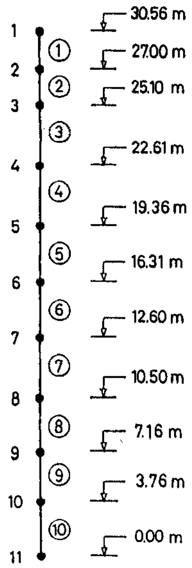
Inertial characteristics

Node	Weight (t)	Inertial moment (m^{2}t)
1	21795	—
2	34223	—
3	13447	—
4	18147	—
5	33270	$3{\cdot}64 \times 10^7$

Beam characteristics

Beam	Cross-sectional area (m^2)	Moment of inertia (m^4)
1	—	3286
2	—	1860
3	196·6	123462·5
4	52·4	62890·5
5	340·0	38694
6	45·75	62834
7	—	500000

Figure 2. Simplified model of reactor building



Inertial characteristics

Node	Weight (t)	Inertial moment (m^2t)
1	1000	—
2	800	—
3	1622	—
4	1986	—
5	2201	—
6	1993	—
7	2143	—
8	1751	—
9	1948	—
10	2114	—
11	6550	10^6

Beam characteristics

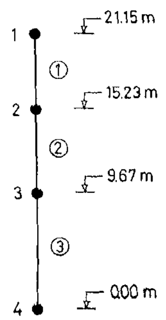
Beam	Cross-sectional area (m^2)	Shear area (m^2)	Moment of inertia (m^4)
1	33·5	25·5	1654
2	33·5	25·5	1654
3	33·5	24·6	1952
4	33·5	24·5	1852
5	34·7	41·5	3577
6	37·9	40·5	3733
7	41·1	41·5	3810
8	41·1	42·8	3830
9	45·5	51·9	4143
10	41·8	28·5	2902

Figure 3. Simplified model of electrical building

building) and Figure 4 (polishing building). In all the analyses the mats and perimeter walls were assumed to behave as rigid structures. A massless box foundation, of the dimensions shown in Figure 1, is also considered in some cases in order to evaluate the influence of kinematic soil–structure interaction.

Seismic excitation

The analyses were carried out for different seismic excitation levels. The response spectra of the accelerograms used were those shown by USNRC Regulatory Guide 1·60, as horizontal and vertical excitation. The supposed peak accelerations as well as the considered response spectra were adopted for final design in some Spanish nuclear power plants. The input accelerograms were specified at the surface of the soil, and a deconvolution process in free field was used to obtain the input motion to apply to the mathematical model. In some cases the input accelerograms were assumed to be located at rock level in order to account for the amplification effects of the backfill layer, and thus its influence on the results. Owing to the FLUSH code characteristics, horizontal and vertical excitations must be considered separately.



Inertial characteristics

Node	Weight (t)	Inertial moment (m^2t)
1	3398	271974
2	4994	476740
3	9930	886826
4	9074	733917

Beam characteristics

Beam	Cross-sectional area (m^2)	Shear area (m^2)	Moment of inertia (m^4)
1	160·6	58·3	16460
2	203·5	101·4	27040
3	287·02	102·27	17180

Figure 4. Simplified model of polishing building

Analyses carried out for Problem 1

Reactor building founded on rock (case 2, Table I). The first analysis considers a maximum horizontal acceleration level of 0·12 g, g being the acceleration due to gravity, defined at the free surface of the backfill layer. The soil pressure distribution against the wall is shown in Figure 5. This is equivalent to a force of 15·60 t/m applied at 5·6 m (0·59 H, H being the wall height) from the bottom of the wall. The second analysis considers a level of vertical excitation of about 0·085 g, defined at the same point as in the preceding analysis. The soil pressure distribution is shown in Figure 6, and is equivalent to a force of 6 t/m applied at 3·9 m (0·41 H)

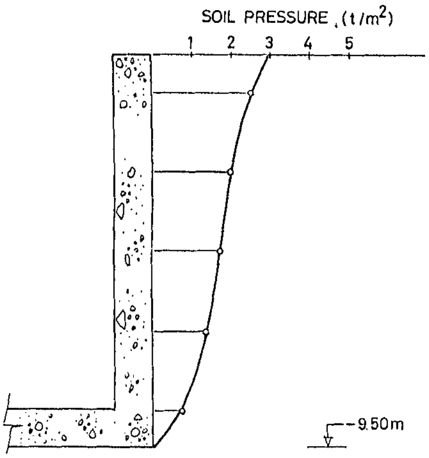


Figure 5. Soil pressure distribution for reactor building on soil media (case 2); horizontal excitation

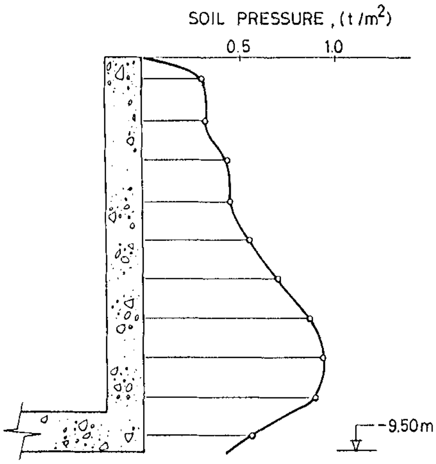


Figure 6. Soil pressure distribution for reactor building on soil media (case 2); vertical excitation

Electrical building founded on rock media (case 2, Table I). A horizontal analysis, similar to the previous ones, was carried out using the electrical building model. None of the changes in soil pressure distribution shown in Figure 5 were observed in this analysis. This important result led to an additional analysis in order to prove that only kinematic soil–structure interaction is significant in this problem. A new analysis was therefore carried out with no structural mass (massless box foundation). Once again, the results were the same, which confirms the above conclusion.

Massless box foundation on hard rock media (case 1, Table I). The results of the analyses for the electrical building prompted two additional horizontal analyses of a massless box foundation for these soil properties. In the first, a horizontal acceleration level of about 0·20 g, defined at the rock–backfill interface, is considered. The soil pressure distribution for this case is shown in Figure 7. The calculated dynamic earth thrust is 70·95 t/m, applied at 5·60 m (0·59 H). The

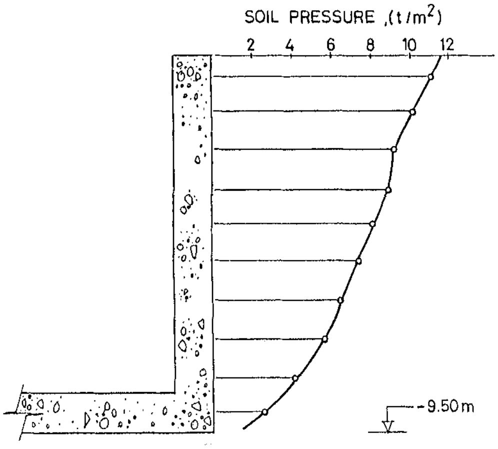


Figure 7. Soil pressure distribution for massless box foundation on soil media (case 1); input motion defined at the rock–backfill interface

maximum horizontal acceleration at the free surface, accounting for the backfill layer amplification effects, becomes 0·42 g. In a second analysis, the maximum horizontal acceleration was taken as 0·25 g at the free surface of the backfill layer. The soil pressure distribution is shown in Figure 8 and is equivalent to a horizontal earth thrust of about 48·06 t/m, with its point of application located at 5·54 m (0·58 H).

Massive box foundation on soft rock media (case 3, Table I). In this last analysis a box foundation with a total weight of about 20,000 t, uniformly distributed along the mat and walls, is subjected to a horizontal earthquake whose maximum acceleration at the free surface is about 0·17 g. The total earth thrust is equal to 32·09 t/m, located at 5·75 m (0·60 H) from the wall base, and with a distribution as shown in Figure 9.

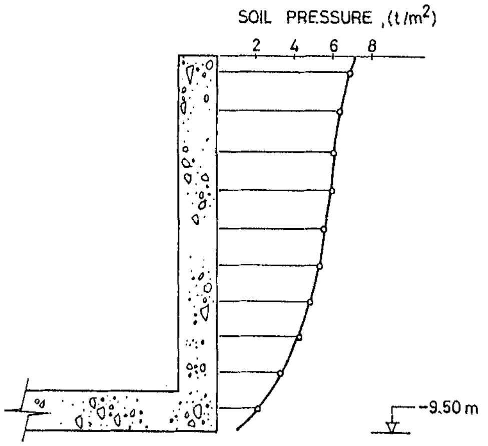


Figure 8. Soil pressure distribution for massless box foundation on soil media (case 1); input motion defined at backfill surface

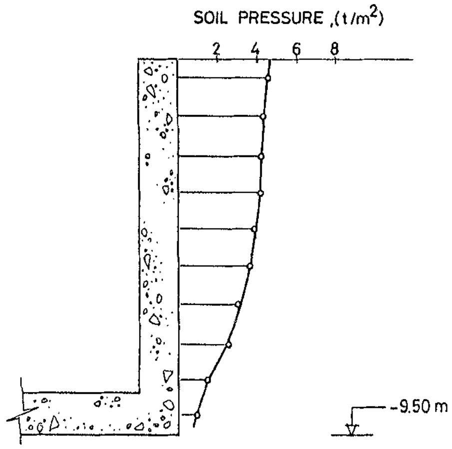


Figure 9. Soil pressure distribution for box foundation (case 3)

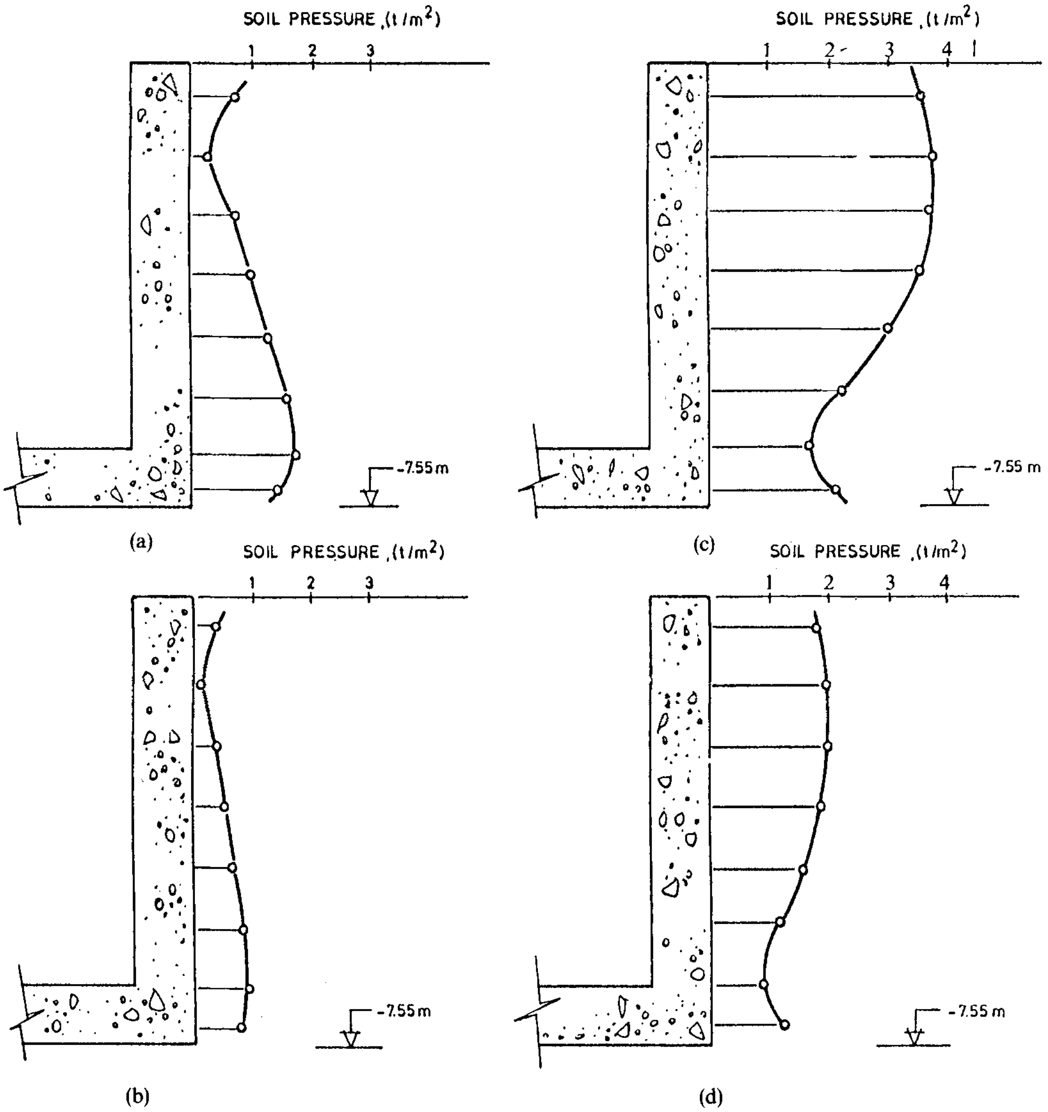


Figure 10. Soil pressure distributions for Problem 2

Analyses carried out for Problem 2

For Problem 2, the structural model used was that corresponding to the polishing building whose characteristics are defined in Figure 4. Four analyses of the problem were carried out: two horizontal and two vertical analyses. The levels of excitation were 0·24 g and 0·13 g, both for horizontal and vertical excitation. These accelerations were assumed to be those at the free surface of the backfill layer.

For vertical analyses the horizontal soil pressure distributions for those excitation levels are shown in Figures 10(a) and 10(b). They are equivalent to 9·24 t/m and 5·06 t/m respectively.

Table III. Horizontal excitation

Horizontal Acceleration ($\times g$)	Computed value (t/m)	MO predicted values (t/m)	Ratio
0·12	15·60	5·60	2·77
0·13	12·67†	3·89	3·26
0·17	32·09	8·38	3·83
0·24	22.8†	8·07	2·82
0·25	48·06	13·47	3·56
0·42	70·95	26·18	2·71

† Problem 2.

Table IV. Vertical excitation

Vertical acceleration ($\times g$)	Computed values (t/m)	MO predicted values (t/m)	Ratio
0·085	6·00	1·67	3·59
0·13	5·06†	1·61	3·13
0·24	9·24†	2·98	3·11

† Problem 2.

For horizontal seismic excitation the horizontal soil pressure distributions for the above-mentioned excitation levels are shown in Figures 10(c) and 10(d); the resultant forces are 22·8 and 12·67 t/m, respectively.

As may be observed, the form of the soil pressure distribution differs very much from those obtained for Problem 1.

Comparison with values predicted by the MO method

In order to compare the computed values with those obtained for the same characteristics of soil and level excitations by the MO method, all the above cases were analysed by this methodology, assuming a value of 40° for the angle of internal friction of the soil.

Tables III and IV show the comparison for both horizontal and vertical seismic excitation.

The values in these tables refer to Problem 1, except those marked by with a dagger, which correspond to Problem 2. The last column gives the ratio between the computed values and those calculated by the MO method.

DISCUSSION OF RESULTS

Two main aspects must be pointed out. The first refers to the great difference between the global results. The MO method seems to underestimate considerably the seismic earth thrust, as reported for previous studies on this subject.[9–11] However, the results obtained here are still more pessimistic (in Reference 9, Dong cites a personal communication from H. B. Seed affecting the dynamic part of earth thrust by a factor of at least two). The present results give an enhancement factor in the range of 2·7–3·8 for the assumed soil conditions and rigid wall behaviour. The numerical technique employed introduces some questionable approximations, which have been discussed in a previous section.

Secondly, it seems that the inertial soil–structure interaction does not greatly affect the results for Problem 1; only the kinematic part needs to be taken into account. In other words, the dynamic earth thrust obtained by the MO method multiplied by an enhancement factor may be used for design. It is interesting to note that a triangular soil pressure distribution is obtained in this paper when the building is founded directly on rock (Problem 1) and the resulting seismic earth thrust is applied at about $0{\cdot}6\,H$. When the building is founded on a backfill layer (Problem 2), the seismic soil pressure distribution depends on whether horizontal or vertical excitation is considered (Figure 10). This confirms that a triangular distribution may not be realistic for a building standing on a backfill soil type.

CONCLUSIONS

From the analyses carried out, the following general conclusions may be drawn: The MO method seems to underestimate the seismic earth thrust, as can be seen if results obtained in the present paper are compared with those obtained by the MO method.

Although numerical results must be interpreted within the framework of the hypotheses and approximations adopted (soil behaviour, body wave action, three-dimensional effects, etc.), it seems reasonable to use an enhancement factor of about three to obtain preliminary design values of the seismic earth thrust against walls of a massive building of a nuclear power plant facility.

For final design, a complete time domain soil–structure interaction analysis is suggested. Additional effects, such as those due to the presence of surface waves, must be introduced into the analyses.

Much more work must be done in this field to obtain enhancement factors that may be applied to values computed by the MO method in order to account for some important effects not included in the present analyses, such as soil plasticity and soil–wall surface discontinuities.

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