

CHAPTER 255

PATRAS BREAKWATER FAILURE DUE TO SEISMIC LOADING

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Abstract

Investigation of the failure mechanism of the Patras breakwater extension revealed that the seismic loading of the mound was amplified considerably due to the soft foundation soil. This loading induced the overriding of the low safety factors of the structure. Approximate analytical expressions were obtained for the hydrodynamic loading during earthquakes. These were in good agreement with previous results. Recommendations for the completion of the works were given to the harbor authority.

INTRODUCTION

Patras is a busy port of western Greece serving as a RoRo gateway to Italy. Its location can be seen in figure 1.

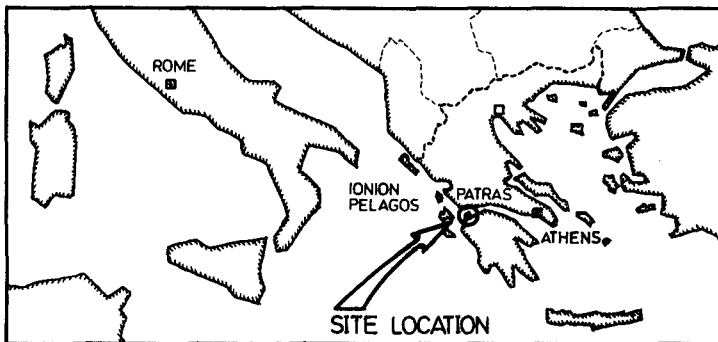
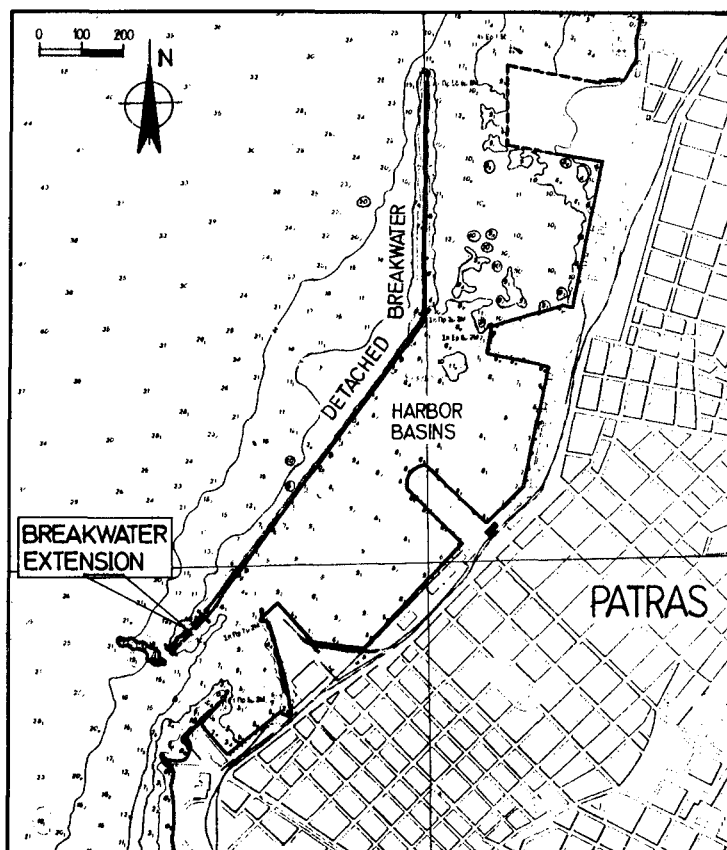


Figure 1. Site location map

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The layout of the harbor is shown in figure 2; it includes a long detached breakwater of rubble mound with concrete capping. During the construction of a southern extension, 120m long, of similar design but without the capping, severe and abrupt settlements of the mound took place leading to a discontinuation of the works.



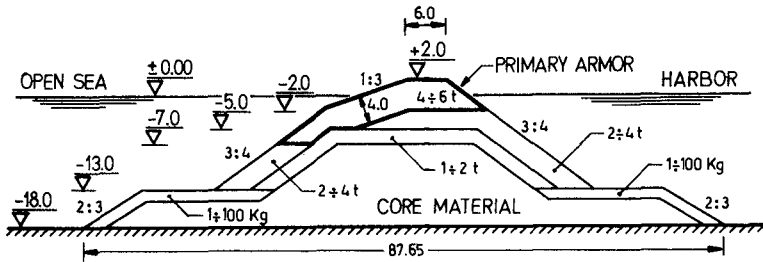


Figure 3. Typical cross-section of the breakwater extension.

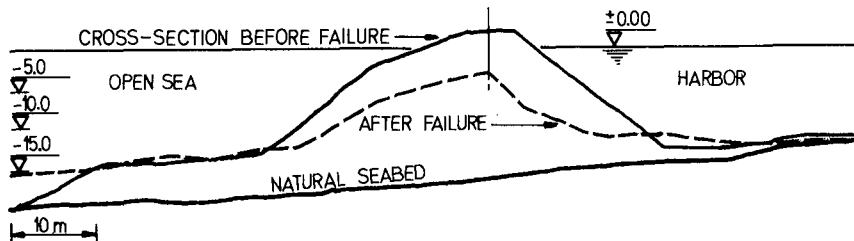


Figure 4. Typical cross-section with outlines before and after the failure

A key element of the present problem was that the failure coincided with moderate seismic activity in the area. Indeed, several earthquakes occurred prior to the major settlement of the structure.

The present research aimed at investigating the dynamic response of the breakwater under construction in order to understand the mechanism that led to its failure. Also, proposals for the completion of the works were to be given to the harbor authority.

The failure mechanism was approached by two complementary points of view, namely hydrodynamic and geotechnical. During the examination of this specific case of failure some more general issues were addressed regarding rubble mounds sitting on soft soils in a seismic area.

In general, the dynamic response of rubble-mound breakwaters has not been so far thoroughly investigated. This can be attributed to the fact that a possible failure of a breakwater entails only a limited reduction of the protection afforded to the harbor. Also, it is usually a matter of routine maintenance to repair such damages by simply adding more stones. For these reasons the current practice of breakwater design is to safeguard stability against wave attack and general soil shear failure, without taking into account the seismic loading. In contrast, the seismic response of similar structures, as earth dams, has been investigated to a far more advanced level, since eventual failure of such structures could well have severe consequences.

An important difference between dams and breakwaters lies in that dams are always designed to sit on firm foundation soils while breakwaters are sometimes

required by other reasons to be constructed on soft soils.

In the following two routes of investigation are presented: the first addresses the hydrodynamic aspects of the problem, i.e. the dynamic loading of the rubble-mound by the surrounding water masses, while the second deals with the seismic analysis of the system "structure-soil".

HYDRODYNAMIC ANALYSIS AND RESULTS

An investigation of the hydrodynamic characteristics of the problem was undertaken aimed at estimating the hydrodynamic loading on the structure due to the seismic activity and assessing thus its relative importance with respect to the "pure" seismic loading acting directly upon the mound through the underlying soil strata.

A simple estimate of the hydrodynamic pressures can be obtained by taking into account the motion of the virtual mass of water in the vicinity of the structure. There are in general two types of modifications to this estimate referring to the compressibility of the water and the elasticity of the structure. Denoting by ω the circular frequency of the horizontal seismic excitation, by $\omega_1 = \pi c / 2h$ the first cutoff frequency of the water body surrounding the structure, c speed of sound waves in water, h water depth, and by ω_s the natural frequency of the structure, the following remarks can be made.

It has been shown by Chopra (1967) that if $\omega < \omega_1$, then in an uncoupled system "breakwater-sea" the compressibility of the water does not play a significant role and can be neglected. In our case we have indeed $\omega < \omega_1$ by feeding the existing data. In a coupled system "breakwater-sea" the required additional condition for ignoring the water compressibility is $\omega_1 / \omega_s > 2$ (Chopra 1968). This again is applicable in the problem under consideration since the site-specific data give $\omega_1 / \omega_s \approx 7$.

Regarding the parameter of the elasticity of the structure, recent research has verified that it produces a significant modification on the pressure distribution along the face of a rigid dam with increasing ratio of ω / ω_1 . Results by other investigators (Mei et al 1979) show that for $\omega / \omega_1 < 0.2$ the deviation of the total hydrodynamic force by assuming rigid structure is less than 15%, while for $\omega / \omega_1 < 0.1$ the difference is negligible, of the order of 1%. In our problem, which represents a typical case of moderate seismic loading, this ratio is about $\omega / \omega_1 \approx 0.05$. Such a low value suggests that as a first approximation the elasticity of the structure can be ignored for the calculation of the total hydrodynamic force. However, this is not necessarily the case when one is interested in the detailed structure of the pressure distribution along the face of the breakwater.

Following these qualitative results the investigation proceeded to estimating the major component of the hydrodynamic loading, namely the added mass pressures on a vibrating sloping face in the sea. The nomenclature of the simplified problem can be seen in figure 5. A part of the seawater in contact with the face of the breakwater produces dynamic loading due to the accelerations involved. The corresponding pressures can be calculated analytically by estimating at every level the breadth b of the water mass that loads dynamically the structure.

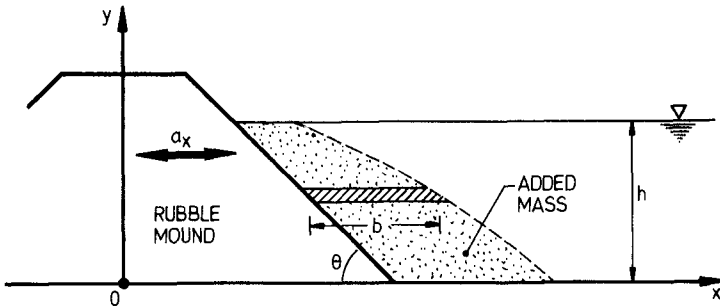


Figure 5. Nomenclature of the simplified problem

An impermeable face was assumed and a no-slip condition was applied at the slope. In figure 6 the external forces acting on a horizontal slice of water mass of height dy are shown.

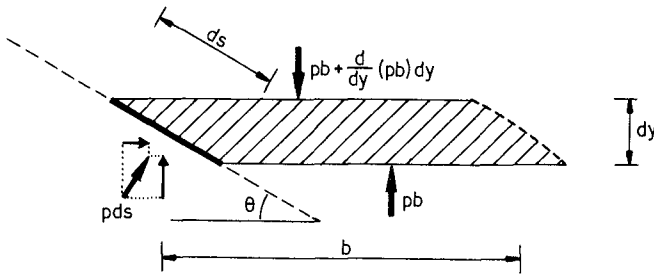


Figure 6. Forces acting on a water element

The equations of motion read,

$$\text{along } y\text{-axis: } p \cot \theta - p - \frac{db}{dy} - b \frac{dp}{dy} = \rho b a_{fy} \quad (1)$$

$$\text{along } x\text{-axis } p = \rho b a_{fx} \quad (2)$$

where a_{fx} , a_{fy} the acceleration of the fluid along x , y axis respectively
 p the pressure

The continuity equation is produced by equating the displaced water volumes due to the motion of the slope during time dt . This yields

$$a_{fy} = a_x y/b \quad (3)$$

The boundary condition along the rigid slope can be written in general,

$$a_{fx} = a_x - a_{fy} \cot \theta \quad (4)$$

On the free surface the boundary condition, taking $p=0$ there, is

$$b = h \cot \theta, \quad y=h \quad (5)$$

Equations (1) to (4) give the following differential equation after some algebra

$$\frac{dp}{dy} = \rho \alpha_x \frac{db}{dy} - \rho b \frac{d\alpha_{xy}}{dy} \cot \theta - \rho \alpha_{xy} \frac{db}{dy} \cot \theta \quad (6)$$

From (3) we obtain:

$$\frac{d\alpha_{xy}}{dy} \cdot b + \alpha_x \sin \theta + \alpha_{xy} \frac{db}{dy} = 0 \quad (7)$$

Now, (6) becomes due to (7)

$$\left(\frac{y}{b} \sin \theta \cot \theta - 2 \right) \frac{db}{dy} - (\sin \theta + \cot^2 \theta) \frac{y}{b} + (1 - \sin \theta) \cot \theta = 0 \quad (8)$$

Integrating eq.(8) with respect to y and employing (5) to define the constant of integration we arrive at

$$\begin{aligned} \log[2b'^2 - b'y'\cot\theta + (1+\cot^2\theta)y'^2]^{1/2} &= (1/2) \log(1+2\cot^2\theta) + \\ &+ \frac{\cot\theta}{H} \arctan\left(\frac{4b'/y' - \cot\theta}{H}\right) - \frac{\cot\theta}{H} \arctan\frac{3\cot\theta}{H} \end{aligned} \quad (9)$$

where the prime denotes non-dimensionalization with respect to h and $H=(8+7\cot^2\theta)^{1/2}$.

If we define a pressure factor $c_p = p/\alpha_y h$ with $\alpha = \alpha_x/g$, then from (2), (3), (4) we obtain

$$c_p = b' - y'\cot\theta \quad (10)$$

The above analytical expression (9) reduces to the following simple formula for the case of a vertical face

$$b' = 0.707 (1-y'^2)^{1/2} \quad (11)$$

This specific result compares very well with existing experimental data of Wang et al (1978) and Zangar (1952) as shown in the graph of figure 7 drawn in terms of c_p .

Apart from the detailed pressure distribution the integrated total force $F=0.56\rho\alpha_x h^2$ falls quite close to other analytical results, deviating only by 5% from the classical value $F=0.58\rho\alpha_x h^2$ given by Westergaard (1933) for the same case of a vertical face.

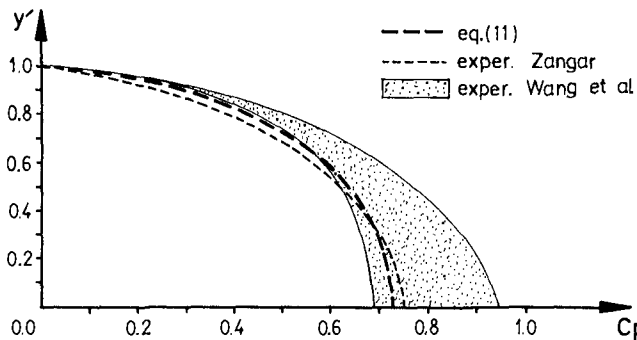


Figure 7. Comparison of analytical expression with experimental results

Application of the general expression to a slope with $\theta = 30^\circ$ gives results that are comparable with experimental data along a central section of the total water depth, but there are deviations from the experimental results at the upper and lower parts of the water column. Nevertheless, the integrated pressure diagram gives again values of the force in good agreement with experimental data exceeding them by about 15%. The deviations of the pressure distribution especially in the lower part of the sloping face can be attributed mainly to the neglected boundary conditions along the sea bed close to the toe of the breakwater. These conditions induce a redistribution of the hydrodynamic pressures on the slope especially to its lower part.

In order to overcome this difficulty a numerical method has been developed which takes into account the conditions along the whole boundary of the water mass. The technique applied to this problem is the boundary integral element method. Variations of the geometry of the boundary, as e.g. sloping sea bed, as well as of its porosity can be accommodated in the model. Due to space limitations this part of the research is not presented here.

The numerical results for the hydrodynamic force based on the previously presented analytical expressions were found to be rather low when compared to the direct seismic loading on the mound through the foundation soil.

The typical cross-section of the breakwater was also checked against wave attack by using a standard method (CERC, 1984). It was found that although in general terms the cross-section was robust, a few modifications could improve considerably its strength, as e.g. extending downwards the seaward armoring. However, such points contributed only secondarily to the initiation of the failure mechanism.

GEOTECHNICAL AND SEISMIC ANALYSES AND RESULTS

Geotechnical Conditions

It was already known from the construction of the older main part of the Patras breakwater and its northern extension that the foundation soil consisted of a rather thick layer of soft compressible clay. Thus prior to the construction of the southern extension, a rather extensive investigation program of its foundation soil was under-

taken. A total number of 21 borings reaching a depth of 50m from the sea level were performed covering a zone 125m long and 100m wide, up to 50m away from the axis of the extension. After the 1984 failure an additional very deep boring was performed to a depth of 104m. Apart from the SPT counts, 1-D consolidation tests as well as drained and undrained triaxial tests were performed on specimens taken from all these borings.

The above in situ and laboratory tests revealed that the foundation subsoil consists of a normally consolidated soft clay layer 30 to 38 meters thick underlain by a thick (>50 meters) moderately overconsolidated medium to stiff clay deposit. Some basic geotechnical parameters of the upper soft clay layer vary as follows:

- Standard Penetration Test Count: $N = 0 \div 15$
- Initial Unit Density: $\rho = 1.75 \div 1.92 \text{ Mg/m}^3$
- Initial Void Ratio: $I = 0.72 \div 1.38$
- Natural Water Content: $W^o = 25\% \div 30\%$
- Plasticity Index: $I = 17\% \div 27\%$
- Compressibility Index: $C^p = 0.20 \div 0.42$
- Undrained Shear Strength: $C_u^c = 5 \div 40 \text{ kN/m}^2$

The lower stiff soil deposit presented the following values for some important geotechnical parameters

- SPT: $N = 42 \div >50$
- Unit Density: $\rho = 1.90 \div 2.00 \text{ Mg/m}^3$
- Void Ratio: $I = 0.69 \div 0.85$
- Compressibility Index: $C^p = 0.13 \div 0.18$
- Undrained Shear Strength: $C_u^c = 120 \div 250 \text{ kN/m}^2$

An idealized soil profile with the selected values of the parameters used in the geotechnical analyses is depicted in figure 8.

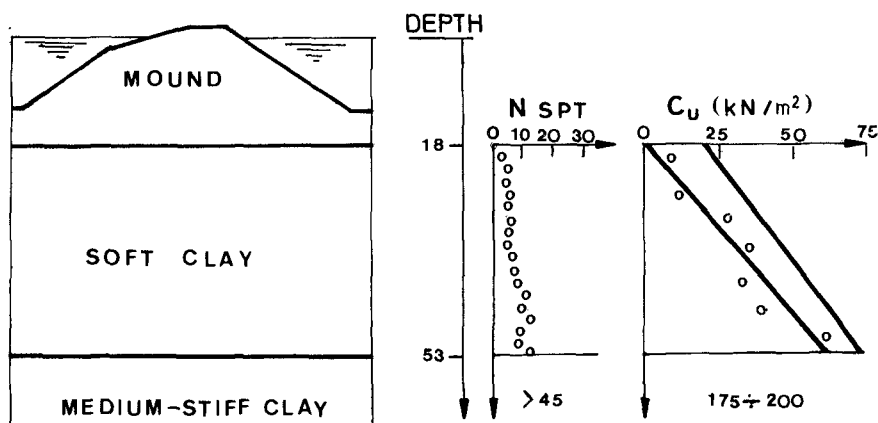


Figure 8. Idealized soil profile and geotechnical parameters

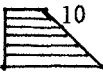
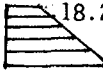
Settlement and Stability Considerations

Settlement and static stability analyses were performed for two stages of construction: one for the phase after the height of the mound had reached the level of 11 meters from its foundation, and a final one for the stage immediately after the completion of construction (height of mound: 18 meters). This was deemed appropriate in order to better simulate the actual history of the construction process, since there was a pause of 6 months after the mound had reached the height of 11 meters, and thus make it possible to account for the changes of the soil parameters caused by the ongoing consolidation process of the upper soft clay layer.

One dimensional settlement analyses yielded as best estimate of the final settlement due to the 11m high rubble mound at its axis approximately 1.5m, whereas the additional settlement due to the rest of the mound was estimated 0.6m, i.e. total settlement 2.1m. The actual settlements was not possible to be measured with accuracy, as they were obscured by the fact that significant amounts of the lower mound material intruded into the soft clay, since no filter zone was provided between the rubble mound and the clay layer. However, the measurements indicated that the total settlement must have been somewhat higher than the above value, approximately 3m; this difference between actual and estimated amount of settlement is mainly due to uncertainties of the clay compressibility parameters estimates, as well as to the limitations of the one-dimensional deformation model considered.

Static stability analyses were performed of the above mentioned two stages of construction. For each stage the undrained shear strength of the upper clay layer was estimated in accordance with the adopted relation $C_u = 0.20 \cdot \sigma'_v$, where σ'_v the effective vertical stress, after a suitable degree of consolidation had been assumed. Two suppositions of the distribution of C_u with depth, uniform and trapezoidal, were considered for each stage. Results produced by the modified Bishop slope stability analysis (Bishop 1955) are presented in table 1.

TABLE 1
STABILITY SAFETY FACTORS FOR STATIC CONDITIONS

CONSTRUCTION STAGE (Height of Mound)	UNDRAINED SHEAR STRENGTH DISTRIBUTION IN THE UPPER LAYER C_u (kN/m ²)	SAFETY FACTOR F
1st Stage (11m)	20 Uniform	1.20
1st Stage (11m)	 10 61.2 Trapezoidal	1.15
2nd Stage (18m)	35 Uniform	1.09
2nd Stage (18m)	 18.2 71.1 Trapezoidal	1.13

It is apparent from these marginal static safety factors that even a moderate dynamic loading might produce failure conditions.

Seismic Response

In late February 1984 a series of moderate earthquakes of magnitude 3.5 to 4.5 took place in the Patras Gulf near the breakwater site. Immediately after these earthquakes settlements of the order of 3 to 4 meters were measured on the constructed part of the southern extension of the breakwater. In this paragraph we investigate the seismic behavior of the breakwater and its foundation during the strongest of these events, which apparently led to its failure. For this purpose, the bedrock ground motion characteristics were estimated and the seismic response of the "rubble mound-soil foundation" system was calculated, using an appropriate one-dimensional model.

Figure 9 shows a map of the Patras area depicting the epicenters of the two strongest earthquakes with $M=4.4$ and $M=4.5$, corresponding epicentral distances 7 and 8 km, and hypocentral distances 9 and 11 km from the site. According to attenuation relationships suitable for western Greece (Papaioannou 1988), the maximum bedrock ground accelerations were estimated at $0.015 \div 0.020g$.

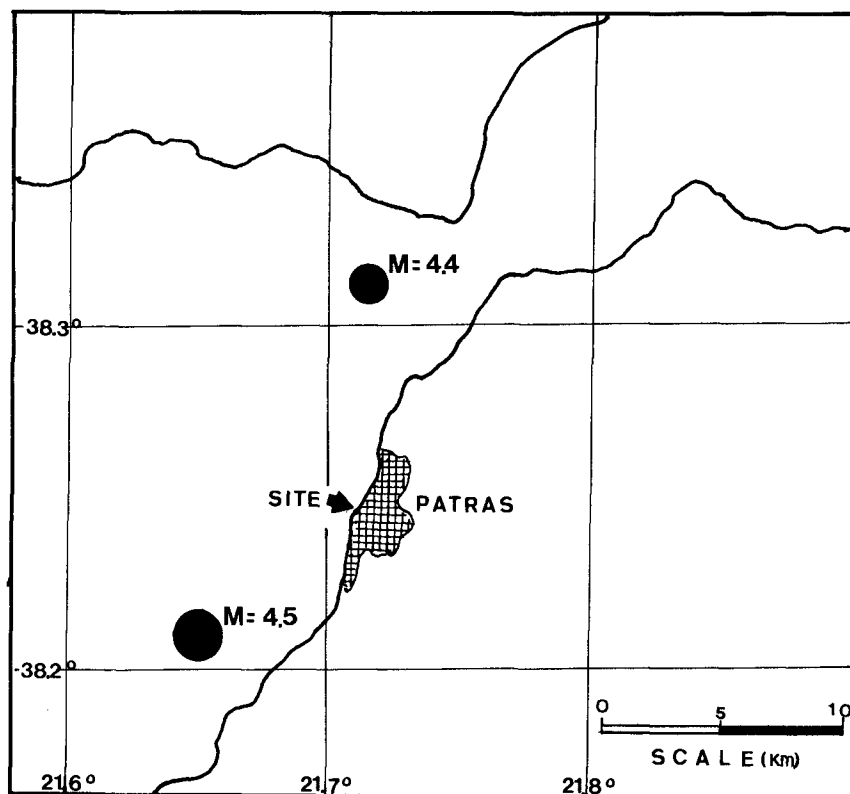


Figure 9. Map showing the location of the epicenters

For the seismic response analysis of the "foundation soil-rubble mound" system as input motions were considered the Kalamata 1986 earthquake record normalized to a peak acceleration of 0.02g (Motion 1) and the Taft earthquake record normalized to a peak acceleration of 0.015g (Motion 2). The first motion was selected because it was recorded at similarly near source site, whereas the second was selected to account for a broader range of fundamental periods.

The seismic response of soil deposit and the rubble mound was calculated by simulating the foundation soils as strata of infinite horizontal extent and the mound as a shear beam (Gazetas 1987). For these analyses the software package SHAKE (Schnabel et al 1972) was used, after a suitable modification. As no dynamic measurements of soil parameters were available, dynamic shear moduli and damping coefficients for the mound and foundation materials were estimated from their density, confining pressure and shear strength characteristics.

Figure 10 shows the variation of initial shear modulus (G_0) and shear wave velocity (V_{s0}) with the depth of the mound and the soil profile.

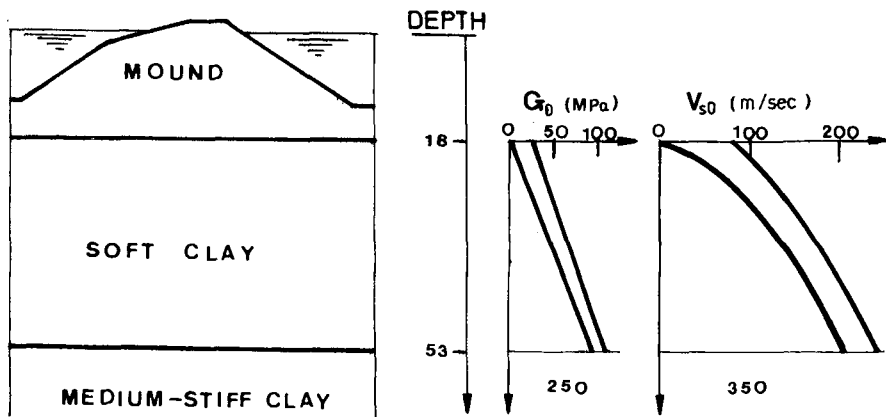


Figure 10. Distribution of dynamic parameters

Results of the analyses as summarized in figure 11 and 12 show that the maximum ground acceleration at the top of the soft stratum was amplified by 2 to 2.5 times. These figures also show that the accelerations were moderately amplified within the mound body.

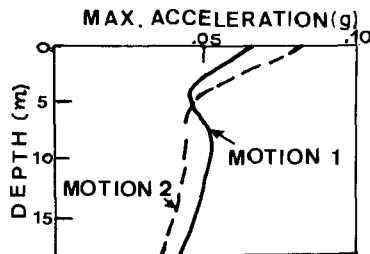


Figure 11. Calculated accelerations along the axis of the mound

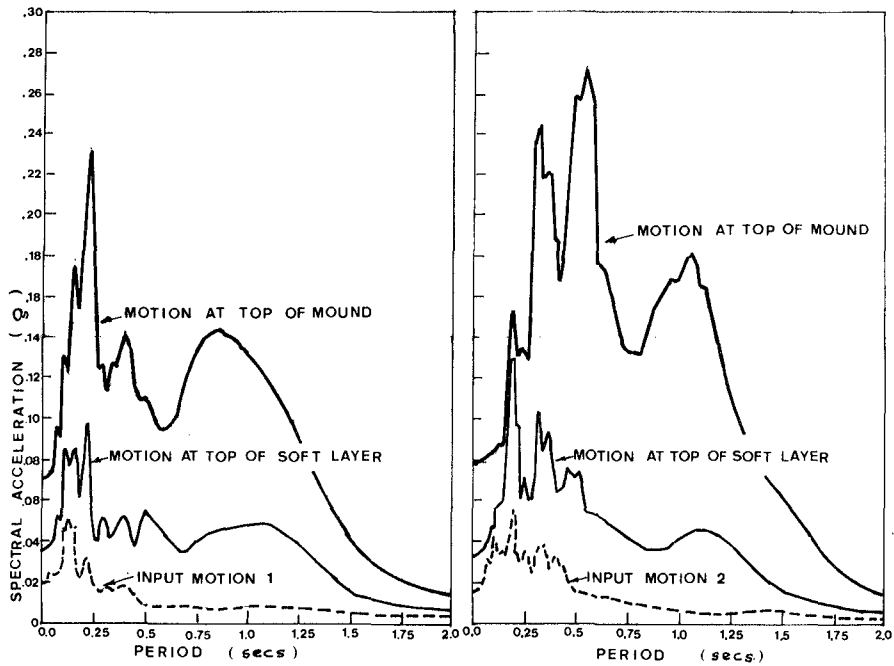


Figure 12. Input and calculated response spectra

Pseudo-dynamic stability analyses were also performed, utilizing the above calculated seismic forces. Results of these analyses are summarized in table 2.

TABLE 2
STABILITY SAFETY FACTORS FOR SEISMIC CONDITIONS

DISTRIBUTION OF UNDRAINED SHEAR STRENGTH FOR THE SOFT CLAY C_u (kN/m ²)			SAFETY FACTOR MOTION 1	SAFETY FACTOR MOTION 2
	35	Uniform	0.82	0.75
	18.2 71.1	Trapezoidal	0.88	0.79

The results show clearly that these relatively small earthquakes were sufficient to trigger the failure mechanism of the structure. The main reason for this was the presence of the deep soft clay stratum that amplified considerably the moderate underground seismic motion, overriding the already marginal static safety factor.

CONCLUSIONS AND RECOMMENDATIONS

Based on the previous analysis of the failure of the Patras breakwater extension the following conclusions can be drawn:

- (1) The failure of the mound was initiated by the seismic activity of February 1984 occurred in the vicinity, which caused the already low safety factors of the structure to fall below an acceptable level. Although the causative earthquakes were rather weak the seismic forces were considerably amplified by the thick soft clay layer on which the mound was founded and caused the shear failure of the low strength foundation.
- (2) The original design needed some improvement to withstand wave attack. However, this played only a secondary role in the failure mechanism.
- (3) The hydrodynamic loading due to the shaking of the mound can be approximated in this particular case by the pressures exerted by the added mass of the water. The structure elasticity as well as the water compressibility can be ignored without affecting appreciably the results.
- (4) The calculated hydrodynamic force due to earthquake activity was small compared to the direct seismic loading on the mound through the foundation soil.
- (5) In general the analytical expression proposed for the hydrodynamic loading on an inclined rigid face is a good first approximation to the total hydrodynamic force and represents an upper bound of the actual load on a porous breakwater face.

The authors believe that this case study represents conditions that can be met in several ports around countries with high seismic activity. Thus a more careful design approach and construction procedure should be followed in such cases. A thorough soil investigation program and a complete seismic analysis included in the design can save unnecessary and costly delays or even failures.

In situations where such adverse conditions are involved, the designer can employ techniques such as: interventions to the geometry of the structure, as e.g. by providing milder slopes or berms; improvement of the foundation soil; use of geotextiles; phasing of progress of works.

Part of the scope of the present research was the proposal of guidelines for the completion of the breakwater extension. In this respect the present geometry of the semi-completed mound, as modified by the wave action over a period of 8 years, has been recorded.

The current layout of the structure as well as standard calculations on armoring against wave attack led us to propose for the trunk section slopes at 1:2 both sides, and for the head of the structure milder slopes at 1:2.5. These new cross-sections were then checked for both static and dynamic stability giving the following factors of safety.

Trunk section

Static conditions	$F = 1.54$
Dynamic conditions ($\epsilon=0.06g$)	$F = 1.18$

Head section

	<i>Short term</i>	<i>Long term</i>
Static conditions	1.42	1.76
Dynamic conditions ($\epsilon=0.06g$)	1.07	1.25

The head section was proposed to be completed in phases, since it has been subjected to a smaller degree of preloading and subsequent improvement by consolidation than the trunk section of the breakwater.

Consideration of other methods of foundation soil improvement, such as sand drains, geotextiles, etc, proved uneconomical for this particular case, mainly due to the small size of the project and the fact that a sizeable part of the structure had already been constructed, making thus interventions of the above kind very costly.

Suitable monitoring to record the behavior of the structure with time has been also proposed.

ACKNOWLEDGEMENTS

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