Effect of Travelling Waves on a Shock Isolated Nuclear Power Plant Put on Rock Foundation

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ABSTRACT

Conventional analyses of the earthquake response of structures are commonly performed with the hypothesis of vertically incident S and P waves (body) by applying a specified ground motion at the base of the model.

Many studies have been performed in order to determine the influence of eventual travelling surface waves. In general, the resulting stresses and displacements are no larger than those encountered in conventional analyses.

As for a Nuclear Power Plant with a conventional foundation, a shock isolated Plant is also calculated with the hypothesis of vertically incident body waves.

However differences between the designs, particularly in view of the large in-plane dimensions of the common raft associated with the shock isolated version, necessitate certain specific considerations.

The object of this study is to estimate the influence of travelling waves on a shock isolated Nuclear Power Plant and to check whether the same safety margin exists for travelling waves as for the body wave assumption.

This comparative study between travelling waves assumption and body waves assumption is performed in both cases of a shock isolated Nuclear Power Plant and of a conventional one (non-isolated) founded on rock.

The analyses give equivalent results for the shock isolated design and for the conventional design.

1. INTRODUCTION

When considering the effects of seismic loadings on the design of structures, soil-structure interaction analyses are performed on the basis that all points at the ground surface move in phase, horizontally and vertically. Such an assumption corresponds physically to vertically propagated waves : S-waves generate the horizontal motion, whereas P-waves generate the vertical motion.

However, during an earthquake, other types of waves are generated, such as surface waves (Love and Rayleigh) and other travelling waves. All these waves combine themselves and, due to multiple reflections and refractions in a soil or rock presenting heterogeneities, and at a ground surface showing mountains and valleys, result in a motion which is random not only in time but also in space, and which unless more data and calculation techniques are available, cannot be handled properly. In other words, we have certain knowledge about the frequency content, duration, attenuation but much less as far as the motion correlation between two adjacent points is concerned :

- With tele-seismic distant epicentres, all generated waves are damped out except long period surface waves. In this case, the motion travels at the ground surface with a given velocity.

- At intermediate epicentral distances, surface waves have been observed as for the San Fernando (9 Feb 71) earthquake owing to the Californian site conditions (active fault, shallow focus, alluvium) (1).

- With very short epicentral distance (a few kilometers), the presence of surface waves is not apparent and if so, they would be totally masked by the tremendous direct body waves emitted from the source, and which represent the main part of the tremor. In this case, the standard assumption of body waves can be considered as adequate. This is the case for most Nuclear Power Plants for which the SSE input data used for design generally corresponds to near-field events.

Due to the lack of data which could support a new design input available for small or intermediate distances and based on another wave propagation assumption, all calculations dealing with any other assumption, such as horizontally travelling waves, must be considered as pure mathematical exercices and the results carefully analysed. Many experts have already analyzed the influence of horizontally travelling waves in comparison with the body waves assumption and concluded that the standard assumption associated with usual material safety factors leads to satisfactory results (2).

The aim of this paper is to present such a comparison performed on a shock isolated (aseismic) structure and on a conventional (non isolated) structure which are designed on the basis of the body waves assumption and to determine the safety margin in the case of travelling waves assumption.

The shock isolated structure uses the aseismic bearing system (neoprene + friction plates) (3)(4).

The site corresponds to a rock site. In order to make a proper comparison, this study is performed with the same seismic input. This input has an horizontal velocity $V = 1600$ m/s (shear wave velocity of the rock) or V = infinite. These two velocities correspond respectively to the travelling and long wave assumption.

The comparison is evaluated for the conventional and shock isolated models by means of relative responses under the travelling wave and body waves.

In a second step, the analysis was developed evaluating the safety margins for both structures under both type of waves.

2. TRAVELLING WAVES - GENERAL

As far as the infinite half space is concerned, one can analyse the propagation of travelling waves (5). The equation of motions of the soil particle is:

$$u = \sin\left[2\pi\left(\frac{t}{T} - \frac{x}{L}\right)\right]\left\{-\frac{2-k_1^2}{2\alpha_2}\exp\left(-2\pi\alpha_1\frac{z}{L}\right) + \frac{1}{\alpha_2}\exp\left(-2\pi\alpha_2\frac{z}{L}\right)\right\}$$

$$v = \cos\left[2\pi\left(\frac{t}{T} - \frac{x}{L}\right)\right]\left\{\frac{2}{2-k_1^2}\exp\left(-2\pi\alpha_1\frac{z}{L}\right) - \exp\left(-2\pi\alpha_2\frac{z}{L}\right)\right\}$$

Where :

t = Time
T = Period
x = Horizontal distance
z = Depth (downward direction positive)
L = Wave length
$k1 = \frac{Vr}{Vs}$ $k2 = \frac{Vr}{Vp}$

Vr = Rayleigh wave velocity
Vs = Shear Wave velocity
Vp = Compression wave velocity
$\alpha 1 = \sqrt{1 - k_1 2}$
$\alpha 2 = \sqrt{1 - k_2 2}$

These equations show that the surface motion is a retrograde ellipse in the vertical plan of the propagating direction. These equations show that the amplitude of the motion decreases with depth, this reduction being particularly great the higher the frequency.

The velocity of propagation of the Rayleigh wave Vr is slightly lower than Vs. The ratio k1 is determined by the Poisson's ratio following the equation :

$$\frac{1}{8}k_1^6 - k_1^4 + \frac{2-\nu}{1-\nu}k_1^2 - \frac{1}{1-\nu} = 0$$

One can evaluate as a function of the hypocenter depth h the minimum epicentral distance which corresponds to the generation of surface waves. This range is further from the epicenter than the minimum distance S_p and S_s (5) :

$$Sp = \frac{k2\ \ h}{\sqrt{1 - k_2 2}}$$

$$Ss = \frac{k1\ \ h}{\sqrt{1 - k_1 2}}$$

ν	.25	.33	.40	.50
Sp	.625 h	.577 h	.416 h	0.
Ss	2.35 h	2.59 h	2.83 h	3.25 h

The case of a horizontally layered soil profile has been also investigated. In this case there are higher Rayleigh modes. The velocity of the Rayleigh waves depends on the frequency. The higher frequency has a velocity corresponding to the first upper layers whereas the low frequency has a velocity corresponding to the deeper layers (7).

3. COMPARATIVE STUDY OF THE RESPONSES OF CONVENTIONAL DESIGN AND ASEISMIC DESIGN

The aim of this section is to evaluate the influence of travelling waves on the shock isolated design by comparison of the responses of two identical structures (group of buildings of the Nuclear Power Plant) one resting directly on the rock, the other put on aseismic bearings using the body wave assumption and the travelling wave assumption.

3.1 Seismic input

On account of the actual properties of the site (postulated fault at 8 km from the site, non shallow focus (in the order of 20 km), a very conservative assumption of travelling waves input consists of using the whole body waves record as a surface wave, travelling at a horizontal velocity v = 1.600 m/s.

In fact, the foundation mat of the structure is rigid in the horizontal direction and it will tend to average the ground motion. When the assumption of the uniformity of the input motion is abandoned, it may lead to a reduction of the translation motion (self cancelling effect). (8) (9)

The body waves inputs are the horizontal and vertical Artificial Time Histories SSE developed for the specific site. The calculations are performed in the time domain because of the non linear behaviour of the aseismic bearing system.

3.2 Mathematical model

The two mathematical models are lumped-mass models (Fig. 1). The first one (Conventional Model) represents two Reactor Containments, their Internal Structures and Nuclear Auxiliary Building on a common raft placed directly on the bedrock. The second one (Shock Isolated Model) represents the same buildings put on aseismic bearings.

- As far as the structural model is concerned, the individual structural stiffness and mass matrices of each building are obtained on the basis of static condensation of complete finite element model and then are set up so as to achieve the total stiffness and mass matrices.
- As far as the bedrock model is concerned, an equivalent infinite half space is derived using a method outlined by Christiano (10). Then stiffness matrices are obtained by inversion of the flexibility matrices calculated by means of Green's functions determined for an harmonically excited point load applied to the half space. Due to the non-linear behaviour of the structure, the analysis is performed in the time domain and consequently there is no possibility of accounting for frequency dependant parameters. The equivalent static stiffness and equivalent damping are adjusted following the frequency of the fundamental modes in the horizontal and vertical directions. As the horizontal and vertical matrices are separated, it is possible to choose adequate frequency in both directions.
- As far as the aseismic bearings model is concerned, the horizontal and vertical stiffnesses of the pads (neoprene + concrete pedestals) can be derived from the horizontal frequency and the vertical deflection d under the weight of the Nuclear Power Plant. The horizontal frequency N is chosen to be about 1 Hz. The friction coefficient of the friction plates is taken $f = 0.25$.

3.3 Methodology

A Nuclear Power Plant in a seismically active region has to be designed so as to withstand seismic load case + non seismic load case. As the seismic response of a shock isolated structure is smaller than that of a conventional structure, the seismic case of loading represents part of the total dimensioning loading for the former, whereas it represents most of these loading for the latter. Then the comparison between both models is made on the basis of total loadings by evaluating amplification factors (ratio between the total sollicitation under the travelling wave assumption and the total sollicitation under the body wave assumption). Evaluating the tension of prestressing cables and of compression and shear of concrete are achieved for different levels in the containment and Internals.

An amplification factor higher than 1.0 means that the travelling waves assumption leads to higher results than the body waves assumption.

3.4. Main results

- The amplification factors in the shock isolated model are sometimes slightly higher than 1.00 (although they remain smaller or comparable with those of the conventional model).

Section	Stress	Amplification factor	
		aseismic	conventional
Containment top (level + 45.00)	Tension in prestressing	1.00	1.01
	Compression in concrete	1.00	1.01
Containment bottom	Tension in prestressing (- 4.50)	1.00	1.11
	Compression in concrete (0.50)	1.00	1.08
	Shear in concrete	1.06	1.31
Internals (- 3.50)	Compression in concrete	0.96	1.00
	Shear in concrete	1.02	0.97

- the 2 % damping ratio horizontal and vertical floor response spectra are computed at the top of the internal structures for the shock isolated and conventional models. They show some differences for both models (due to the excitation of the rocking modes by the travelling waves input).
- For a few points design values could be exceeded in both cases. The analysis of the safety margin domain was then developed.

4. SAFETY ANALYSIS

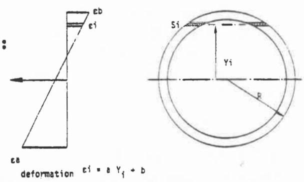
Due to the arbitrary nature and doubt on the existence of such waves at small epicentral distances and to the unreliability of the arbitrary assumptions, such a study introduces conservative results. Consequently the values obtained should not be compared with the design limits, but should at the extreme be considered as values resulting from "damage studies" : damage study is an analysis performed to evaluate what happens beyond the design limits. These types of studies are made in the plastification or Ultimate Limit State domains.

Ultimate Limit State Analysis is used here. A global verification and a shear verification are performed for two critical areas (Base of the Containment and of Internal Structures). This consists of taking the Plant design (concrete, reinforcement, prestressing), evaluating the safety domain of the Plant and plotting on the same diagram representative points of SSE conditions (body waves and travelling waves conditions (v = 1 600 m/s).

4.1 Global verification - Interaction diagram

- Let a section be submitted to a normal force N and a moment M. The static equilibrium will be achieved after a deformation of this section. The basic assumption consists in a linear deformation of the section : the plane surface remains plane.

- N and M are in equilibrium by stresses in concrete and steels :



$$N = \sum_i \sigma c(\varepsilon i).Si + \sum_i \sigma_{s_1}(\varepsilon i).Si.\alpha_1 + \sum_i \sigma_{s_2}(\varepsilon i).Si.\alpha_2 + \dots$$

$$M = \sum_i \sigma c(\varepsilon i).Si.\ i + \sum_i \sigma_{s_1}(\varepsilon)Si.Yi.\alpha_1 + \sum_i \sigma_{s_2}(\varepsilon i)Si.Yi.\alpha_2 + .$$

$\varepsilon c(\varepsilon i)$, $\sigma s_1(\varepsilon i)$, $\sigma_{s_2}(\varepsilon i)$ are the stress/strain relationships of concrete and steels (we can have different kinds of steels in the section) (Fig.2). $\alpha 1$, $\alpha 2$... are the steel/concrete surface ratio.

- The ultimate strains accounted for are :
  tension : steel 1 %
  compression : concrete 0.2 % (in the case of thin cylinders)

There are 3 kinds of stress/strain relationships for the steel in the Containment (Liner, Rebars, Prestressing...) Only the rebars are used for the Internals. The concrete parabola-rectangle curves are given for both Structures (Fig 2). These results are presented for each section and for the different load cases :

(1): Normal Operating (NO)
(2): NO+LOCA (pressure effect)
(3):NO+LOCA+SSE
(4):NO+LOCA+Travelling waves (1600 m/s)

The diagram showing sets (M,N) for which the strain εa or εb reaches the ultimate values of concrete or steel is called the interaction diagram.
The LOCA (pressure effect) has no effect on the Internals. From these results, it turns out that the SSE and travelling waves load cases are close to and located well inside the interaction diagram (Fig 4, Fig 5).

4.2. Shear verification

For each previous load case one can compute for each point of a section the horizontal compression σ'_y the vertical compression σ'_x and the shear stress τ. σ'_y depends on the horizontal prestressing and pressure effect of the LOCA.

The safety domain for the(σ'_x , τ)set values is defined on the basis of the French I.P. 2 recommendations (11) :

$$\tau^2 \leqslant (0.25)^2 . \left[1 + \frac{\sigma'x + \sigma'y}{\sigma'j}\right]^2 \left[\frac{\sigma'j}{\gamma b} - \sigma'x - \sigma'y\right].\left[k\sigma j + \sigma'x + \sigma'y\right] + \sigma'x.\sigma'y$$

- The calculations are performed for different angular values of the circular section at the bottom of the containment in the NO case of loading and NO + LOCA case of loading.
  The results are always located well inside the safety domain. (Fig 6, Fig 7).

- As far as the Internal Structures are concerned, the horizontal compression is zero (no horizontal prestressing nor LOCA pressure effect).

  The vertical compression is calculated from the NO + seismic cases of loading (M, N). The shear stress is induced at the very bottom of the Internals by the shear force T and the LOCA.

  The points representative of SSE conditions and travelling waves conditions are located on the (τ,σ) diagram. Three safety domains are evaluated using different rules (IP 2, CCBA 68, BAEL, ...) and included the previous points. (11, 12, 13).

5. CONCLUSION

The conventional assumption of seismic analysis is based on independant horizontal and vertical ground motions. As it is not possible to identify the contribution of each type of wave, an arbitrary assumption for evaluating the effect of travelling waves is taken. It consists of applying the total seismic motion as a surface motion, which travels at the shear wave velocity. For a Nuclear Power Plant sited on rock, the travelling wave assumption in comparison with the body waves assumption leads to equivalent results, for the shock isolated and for the conventional design. Although a few travelling wave results exceed the design values of the body waves assumption, they remain well inside the safety domain defined on the basis of the Ultimate Limit State. For a more refined evaluation of the effect of the travelling waves phenomenon, it will be necessary to correctly identify the ratio between the body waves and the travelling waves, in order to limit the conservatism of the travelling waves input.

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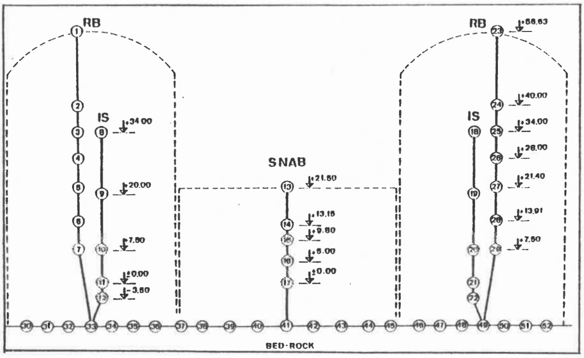


Fig.1 : Conventional Model

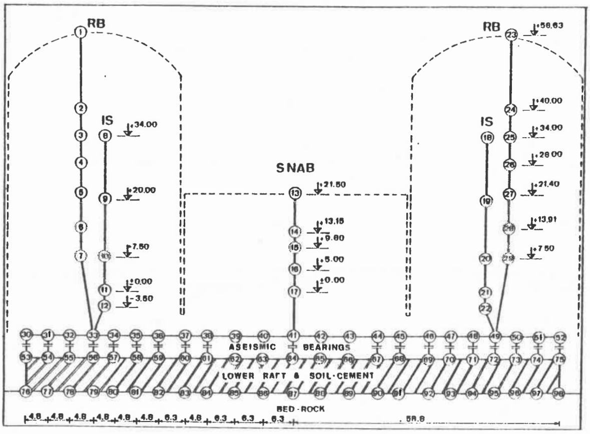
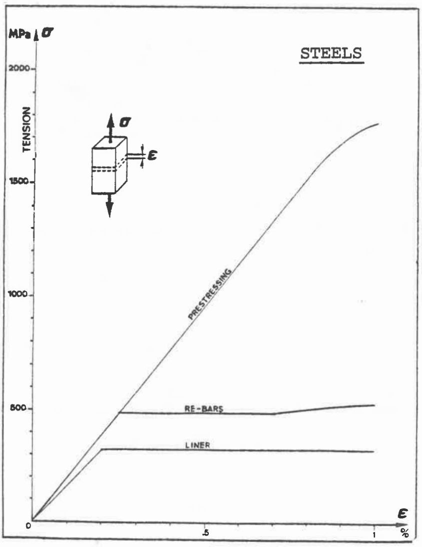


Fig.1b : Shock Isolated Model



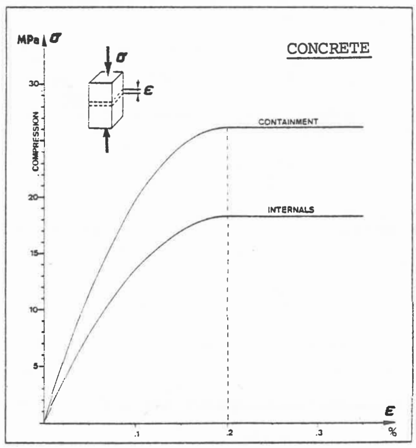
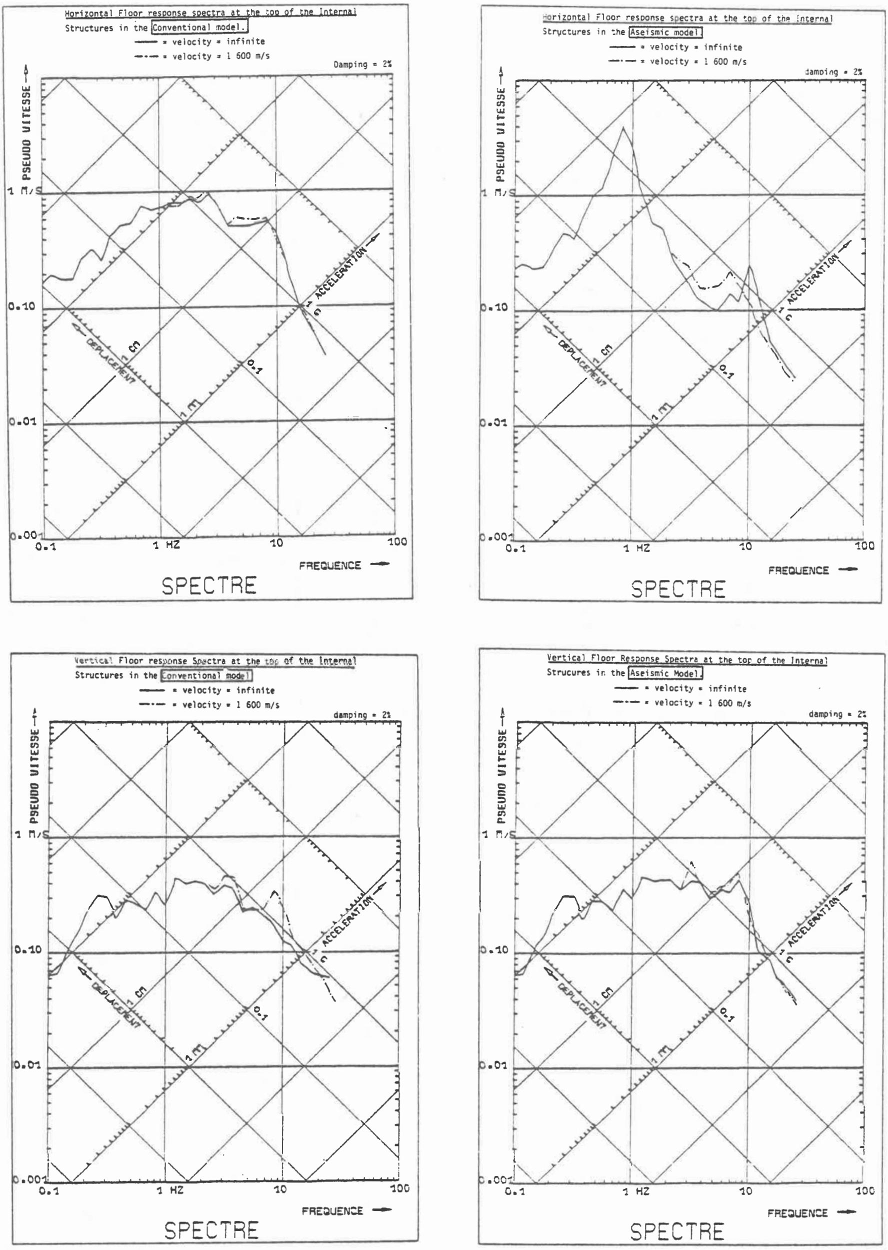


Fig.2 : Stress/Strain relationships of steels and concrete



CONVENTIONAL MODEL

SHOCK ISOLATED MODEL

Fig.3 : Horizontal and Vertical Floor Response Spectra at top of Internal Structures for Conventional and Shock Isolated Models.

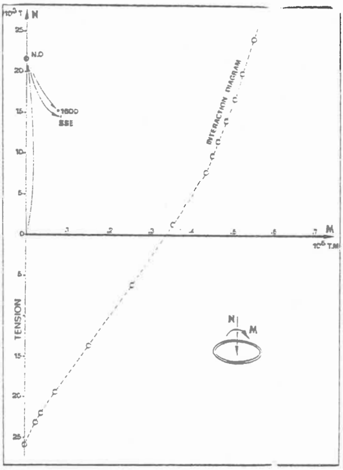


Fig. 4 : Internal Structures
Level -3.50
Global Verification

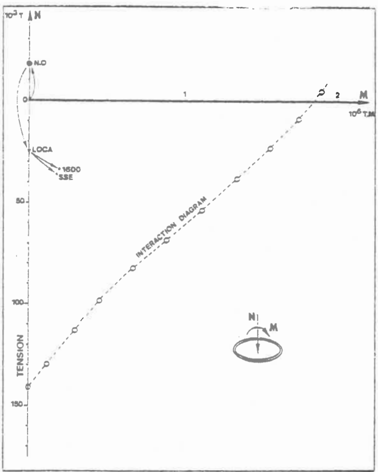


Fig. 5 : Containment Level -0.50
Global Verification

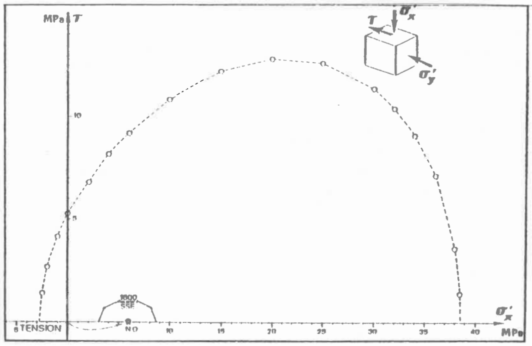


Fig. 6 : Containment - Level -0.50
Shear Verification (N.O.)

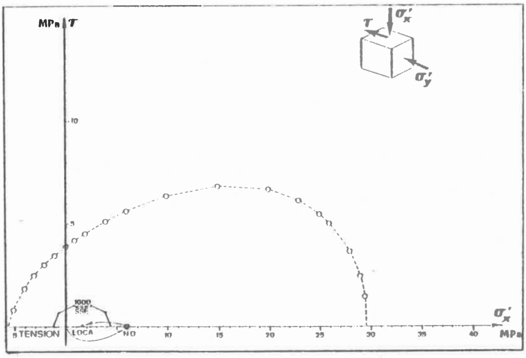


Fig. 7 : Containment Level -0.50
Shear Verification (N.O. + LOCA)

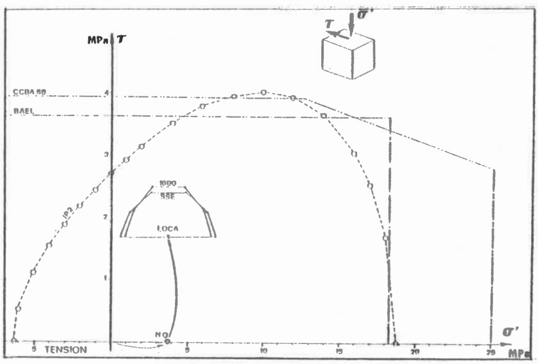


Fig. 8 : Internal Structures Level -3.50
Shear Verification (NO + LOCA)