Load And Resistance Factor Design Foundations Structures

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INTRODUCTION

The goal of this paper is to obtain a set of load and resistance factors that ensure a uniform reliability for the global stability of the different buildings at the Atucha II Nuclear Plant (Argentina).
The buildings are partially buried and lean on reinforced concrete mats that lie on different preconsolidated clay soil.
The usual practice of verifying the foundation load capacity and the overturning safety by means of a global safety factor, leads, as it will be then demostrated to a non-uniform reliability of the structures according to each design condition. Moreover, the subjective way in which these global factors have been determined results in a reliability not always compatible with the one expected at a Nuclear Plant.
In order to develop design criteria yet with a deterministic format, probabilistic methods may be used showing in a rational way the uncertain nature of the design parameters (Ellingwood et al, 1982; Galambos et al, 1982, Hwang et al, 1987).

PROBABILITY-BASED DESIGN CRITERIA

The procedure adopted to obtain a set of load and resistance factors was the following:

1. Define the limit states to consider $g(X_i)= 0$, the random variables involved X_i, mean values μ_i, coefficient of variation δ_i and distribution funtion $F_{X_i}(x)$.
2. Obtain the limit state probability according to the usual design equation using the "Advanced First Order Second Moment Method" (Ang-Tang, 1984), for the different design conditions of the Nuclear Plant buildings.
3. Compare the former probability with the reliability levels usually accepted for the other limit states at the Nuclear Plant designs.
4. According to 2. and 3. select a target limit state probability.
5. Choose a set of partial safety factors that minimize the difference between the target limit state probability and the limit state probability for the expected design conditions.

PROBABILISTIC MODEL OF LOADS

Dead load (G) - Dead load arises from the weight of elements composing the structure, the weight of attachments such as piping, HVAC ducts and cable trays, and the weight of permanent equipment that is not treated explicitly as equipment load.
The dead load intensity is modeled by a normal random variable with a mean value equal to the nominal design value and a coefficient of variation equal to 0,10.

Live load (P) - The live load in nuclear power plants denotes temporary loads and their effect resulting from occupancy, movable equipment, and other operation or maintenance conditions.
Owing to the fact that the probability of live load simultaneous action in various rooms of the buildings is very low, mean value P = 0,10G was considered, with a coefficient of variation of 0,25 and a distribution function of Extreme type I.
Overturning load (M) - Atucha Power Plant is not located in a sismic area, therefore the most important horizontal action is the pressure wave derived from the chemical explotion because of gas traffic in the nearby river.
The mean rate of explotion ocurrence on the buildings is about $\lambda = 10^{-7}$ per year. Its intensity is modeled by a normal random variable with a mean value equal to the nominal design value and a coefficient of variation of 0,25.

PROBABILISTIC MODEL OF RESISTANCE

The mean values of internal friction (φ) and cohesion (c) parameters come from the soil investigation for the different layers on which the nuclear plant buildings lie (Table 1).
The coefficients of variation result from a balance between bibliography (Mc Guffey, 1982; Whitman, 1984; Hoeg, 1974) and soil investigation, with $\delta_c = 0,25$; $\sigma_\varphi = 2°$ beeing adopted with distribution functions log-normal.
The range of mean values of the overload ($\bar{q}$) depends of the different depths in which the power plant buildings are settled (Table 1).
The overload ($\bar{q}$) and the unit weight of soil ($\bar{\gamma}$) are considered as normal random variables with coefficient of variation 0,10.

TABLE 1 - MEAN VALUES AND STATISTICAL INFORMATION

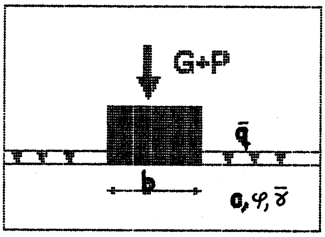
VARIABLE	X_i	μ_i	δ_i	$F_{xi}(x)$	Obs.
φ	X1	5° - 20°	$2°/\varphi$	LOG-NORMAL	
c	X2	20-120 $\frac{KN}{m2}$	0,25	LOG-NORMAL	
$\bar{q}$	X3	50-150 $\frac{KN}{m2}$	0,10	NORMAL	
$\bar{\gamma}$	X4	$\frac{8KN}{m3}$	0,10	NORMAL	
$\frac{G}{bL}$	X5	DESIGN EQUATION	0,10	NORMAL	
$\frac{P}{bL}$	X6	$0,10\mu_5$	0,25	EXTREME TYPE I	with e/b = 0
b	-	10-50m	-	-	DETERMINISTIC VARIABLE
$\frac{M}{Lb^2}$	X6	$\frac{e}{b}\mu_5$	0,25	NORMAL	with e/b > 0

LIMIT STATES

Due to the massive feature of the buildings, only significant relative excentricities take place with accidental loads. Therefore, a limit state with relative excentricity $\frac{e}{b} = 0$ for normal loads and another limit state with $\frac{e}{b} > 0$ for accidental load are considered.

RELATIVE ECCENTRICITY $\frac{e}{b} = 0$

According to Brinch Hansen the limit state of global stability results from the equation.



$$\tfrac{1}{2}\bar{\gamma} b N_\gamma(\varphi) + (c+\bar{q}\,tg\varphi)Nc(\varphi) + \bar{q} - \frac{G}{bL} - \frac{P}{bL} = 0$$

$$N_\gamma(\varphi) = 1,8\ [Nq(\varphi)-1]tg\varphi$$

$$Nc(\varphi) = [Nq(\varphi)-1]cotg\,\varphi$$

$$Nq(\varphi) = e^{\pi tg\varphi}\ tg2\ (45+\varphi/2)$$

Shape factors S_γ and S_c and depth factors d_γ and d_c are not considered for the purpose of this paper.

Failure equation may be rewritten defining X_1, to X_6 values according to table 1:

$$g\ (X_1,\ X_2,\ X_3,\ X_4,\ X_5,\ X_6) = 0$$

$$\tfrac{1}{2}X_4 b N_\gamma(X_1) + (X_2 + X_3 tg X_1)\ N_C\ (X_1) + X_3 - X_5 - X_6 = 0$$

The usual design equation is:

$$\tfrac{1}{2}\mu_4 b N_\gamma(\mu_1) + (\mu_2+\mu_3 tg\mu_1)\ N_c\ (\mu_1) + \mu_3 \geq (\mu_5+\mu_6)\ F$$

F=2, safety factor

The wanted design equation is:

$$\tfrac{1}{2}\ X_4^* b\ N_\gamma(X_1^*) + (X_2^* + X_3^*\ tg\ X_1^*)\ Nc\ (X_1^*) + X_3^* \geq X_5^* + X_6^*$$

in wich:

$$X_1^* = \mu_1 - \emptyset_1 \qquad\qquad X_i^* = \emptyset_i\,\mu_i\ ,\ i=2,\dots,6$$

The reliability index is studied in a parametric way using table 1 values and the mean value of $X_5 = \frac{G}{bL}$ calculated with a) Usual design equation (Fig.1a, 1b)

$$\mu_5 = \frac{\tfrac{1}{2}\mu_4 b N_\gamma\ (\mu_1) + (\mu_2 + \mu_3 tg\mu_1)\ N_c\ (\mu_1) + \mu_3}{(1 + P/G)\ F}$$

and b) Design equation with partial factors

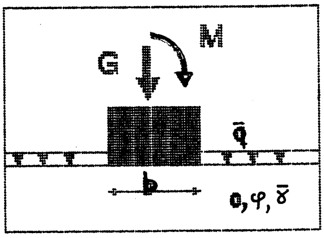
$$\mu_5 = \frac{\tfrac{1}{2}\mu_4 b N_\gamma(X_1^*) + (X_2^* + X_3^*\ tg\ X_1^*)\ N_c(X_1^*) + X_3^*}{(1 + \frac{P\emptyset_6}{G\emptyset_5})\ \emptyset_5} \qquad \text{(Fig. 2a, 2b)}$$

with:

$\emptyset_1 = 4°$	$\emptyset_3 = 0,90$	$\emptyset_5 = 1,10$
$\emptyset_2 = 0,60$	$\emptyset_4 = 1,0$	$\emptyset_6 = 1,60$

RELATIVE ECCENTRICITY $\frac{e}{b} > 0$

According to Brinch Hansen the limit state of global stability results from the equation:



$$\frac{1}{2}\bar{\gamma}(b-2e)N_\gamma(\varphi) + (C+\bar{q}\,tg\varphi)N_c(\varphi) + -\frac{G}{(b-2e)L} = 0$$

$$e = \frac{M}{G}$$

Shape factors S_γ and S_c, depth factors d_γ and d_c and inclination factors i_γ and i_c are not considered for the purpose of this paper.

Failure equation may be rewritten defining X_1 to X_6 values according to table 1:

$$\frac{1}{2}X_4 b\left(1-2\frac{X_6}{X_5}\right)N_\gamma(X_1) + (X_2+X_3\,tgX_1)\,N_c(X_1) + X_3 - \frac{X_5}{1-2\frac{X_6}{X_5}} = 0$$

The usual design equation is:

$$\frac{1}{2}\mu_4 b\left(1-2\frac{\mu_6}{\mu_5}\right)N_\gamma(\mu_1) + (\mu_2+\mu_3 tg\mu_1)\,N_c(\mu_1)+\mu_3 \geq \frac{\mu_5}{\left(1-2\frac{\mu_6}{\mu_5}\right)}\,F$$

F= 1,2 is usually taken for accidental load.

The wanted design equation is:

$$\frac{1}{2}X_4^* b\left(1-2\frac{X_6^*}{X_5^*}\right)N_\gamma(X_1^*) + (X_2^*+X_3^* tg\,X_1^*)N_c\,(X_1^*) + X_3^* \geq \frac{X_5^*}{1-2\frac{X_6^*}{X_5^*}}$$

In wich: $X_1^* = \mu_1 - \phi_1$ $X_i^* = \phi_i\mu_i$, i=2,...,6

Two equations must be checked since G is unfavourable for little $\frac{e}{b}$ and favourable for large $\frac{e}{b}$.

One of them with $\phi_5 < 1$ and the other with $\phi_5 > 1$.

The reliability index is studied in a parametric way using table 1 values and the mean value of $X_5 = \frac{G}{bL}$ calculated with a) Usual design equation (Fig. 3a-3c)

$$\mu_5 = \left[\frac{1}{2}\mu_4 b\left(1-2\frac{e}{b}\right) N_\gamma(\mu_1)+(\mu_2+\mu_3 tg\mu_1)N_c(\mu_1)+\mu_3\right]\frac{1-2\,e/b}{F}$$

and b) Design equation with partial factors (Fig.4a-4c)

$$\mu_5 = \left[\frac{1}{2}X_4^* b\left(1-2\frac{e}{b}\frac{\phi_6}{\phi_5}\right)N_\gamma(X_1^*)+(X_2^* + X_3^*\,tg\,X_1^*)\,N_c\,(X_1^*) + X_3^*\right]\frac{1-2\frac{e}{b}\frac{\phi_6}{\phi_5}}{\phi_5}$$

with:

$\phi_1 = 1°$	$\phi_3 = 1,0$	$\phi_5 \begin{cases} 1,05 \\ 0,95 \end{cases}$
$\phi_2 = 0,9$	$\phi_4 = 1,0$	$\phi_6 = 1,10$

CONCLUSIONS.

1. Levels of reliability implied by the use of current design practice for foundations structures are about $P_f \simeq 10^{-3}$ for normal loads and $P_f = T\lambda p \simeq 40\times10^{-7}\times3\times10^{-1} \simeq 10^{-6}$ for accidental loads considered on a 40-year basis. 2. The reliability of existing design criteria varies according to soil, failure mode, load combination and specially with the relative eccentricity e/b. 3. The LRFD format appears acceptable from the stand point of enabling essentially constant reliability levels to be attained for most likely design situations with additive partial resistance factor for φ and multiplicative factor for the other variables.

These results are preliminary in nature and require further validation prior to general implementation.

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RELATIVE ECCENTRICITY e/b = 0

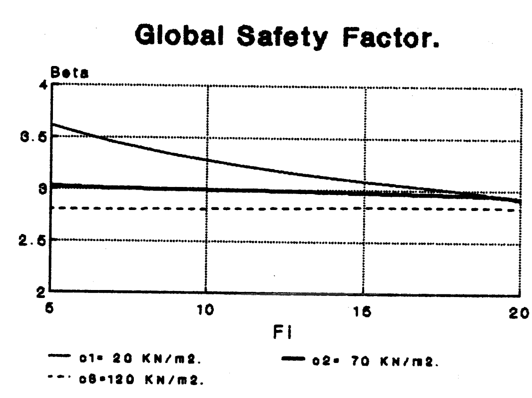

Fig. 1a

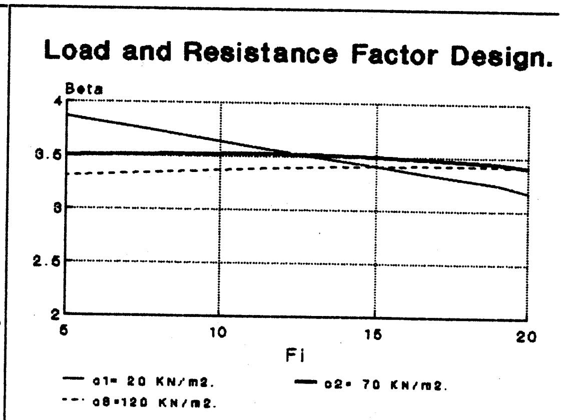

Fig.2a

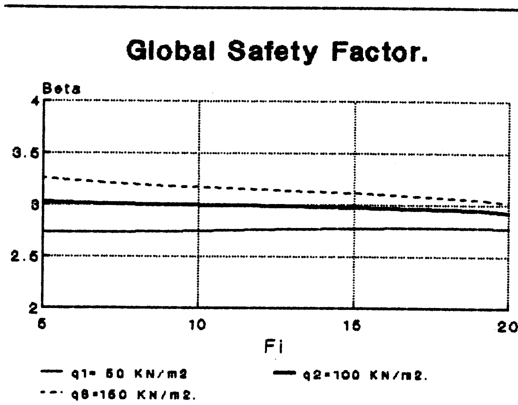

Fig. 1b

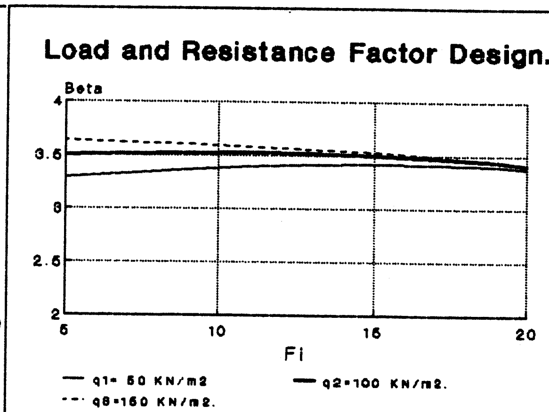

Fig. 2b

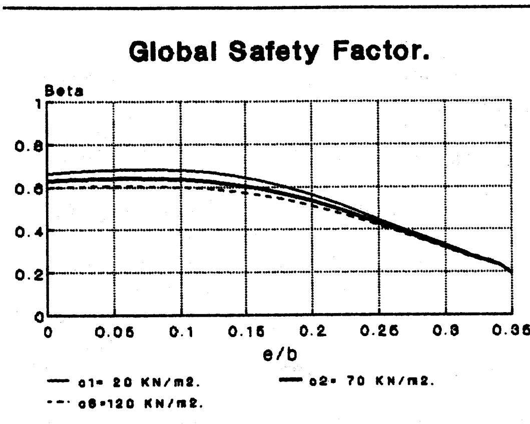

Fig. 3a

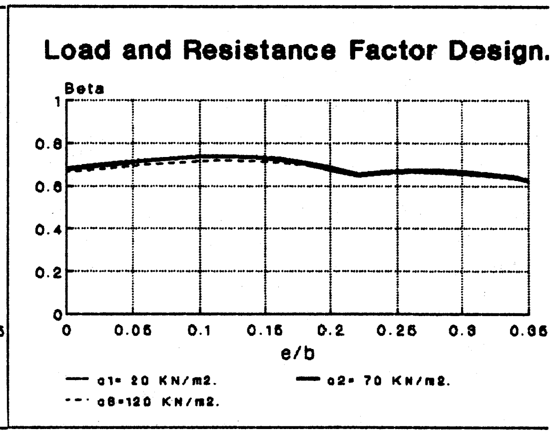

Fig. 4a

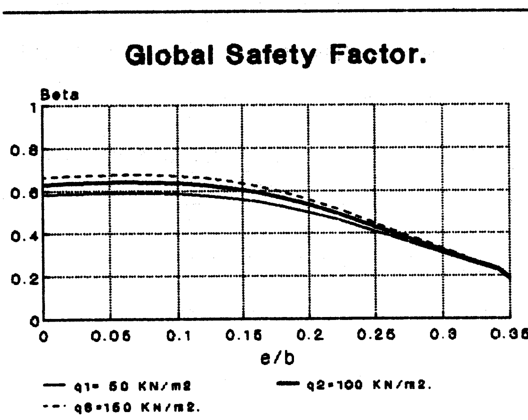

Fig. 3b

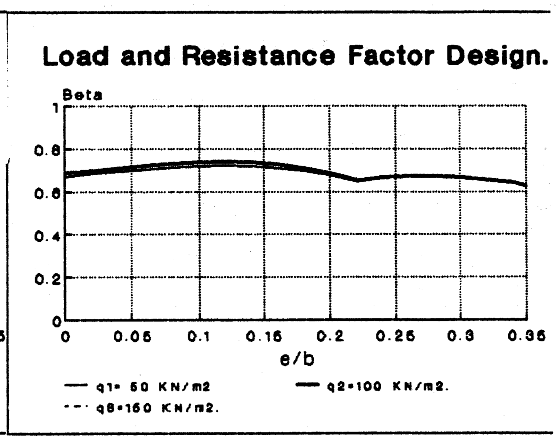

Fig. 4b

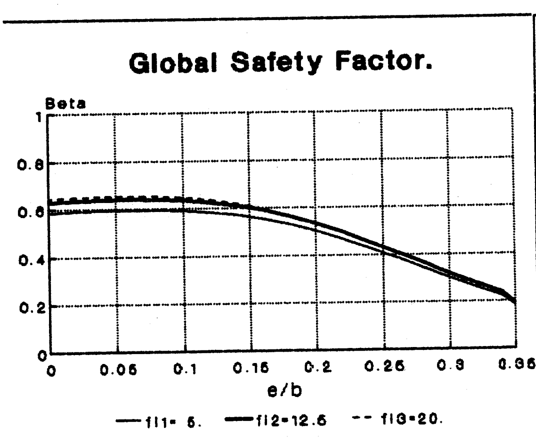

Fig. 3c

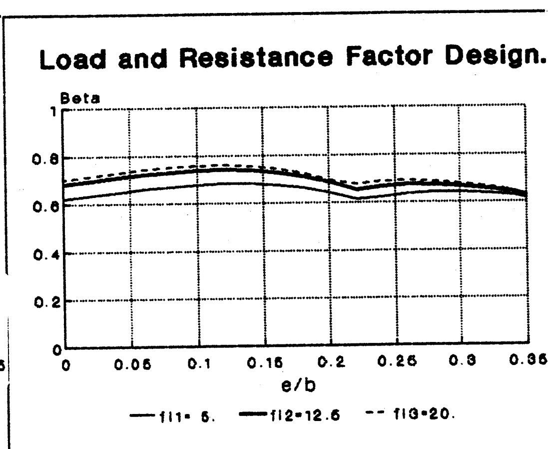

Fig. 4c