*Transactions of the 17th International Conference on Structural Mechanics in Reactor Technology (***SMiRT 17***)*
Prague, Czech Republic, August 17 –22, 2003

Paper # K05-4

Seismic Fragility of Ventilation Stack of Nuclear Power Plant

Nefedov S.S. [1)], Yugai T.Z. [1)], Kalinkin I.V. [2)], Vizir P.L. [2)]

[1)] ROSENERGOATOM Company, Design branch, Moscow, Russia
[2)] Institute ATOMENERGOPROEKT, Moscow, Russia

ABSTRACT

Fragility study of safety related elements is necessary step in seismic PSA of nuclear power plant (NPP). In present work fragility was analyzed after the example of the ventilation stack of NPP. Ventilation stack, considered in present work, is a separately erected construction with height of 100 m made of cast-in-place reinforced concrete.

In accordance with IAEA terminology fragility of element is defined as conditional probability of its failure at given level of seismic loading. Failure of a ventilation stack was considered as development of the plastic hinge in some section of a shaft. Seismic ground acceleration a, which corresponds to failure, could be defined as limit seismic acceleration of ventilation stack $[a]$.

Limit seismic acceleration $[a]$ was considered as random value. Sources of its variation are connected with stochastic nature of factors determining it (properties of construction materials, soils etc.), and also with uncertainties of existing analytical techniques. Random value $[a]$ was assumed to be distributed lognormally. Median $m_{[a]}$ and logarithmically standard deviation β of this distribution were defined by ‘scaling method’ developed by *R.P Kennedy* et al. Using this values fragility curves were plotted for different levels of confidence probability.

KEY WORDS: ventilation stack, seismic action, probability, failure, lognormal distribution, median, standard deviation.

INTRODUCTION

In accordance with IAEA terminology [1] fragility of an element of NPP was considered as conditional probability of its failure P_0 at the given level of seismic action. This level could be characterized by seismic ground acceleration a, which corresponds to ordinate of a of seismic action spectra at zero period of natural oscillations. By such approach seismic fragility could be assumed as a function of seismic ground acceleration $P_0(a)$. Graphically this function could be presented as a family of fragility curves $P_0(a,Q)$ plotted for different values of confidence probability Q.

In present work fragility was analyzed after the example of the ventilation stack of NPP (Fig.1). Last earthquakes show that failure of stack can seriously amplify seismic damage for industrial enterprises of different types. Ventilation stack considered in present work is a separately erected construction with height of 100 m made of cast-in-place reinforced concrete.

METHOD OF ANALYSIS

Failure of a ventilation stack was considered as exhaustion of its bearing capacity and development of plastic hinge in some section of a shaft. Seismic ground acceleration a, which corresponds to failure, may be considered as limit seismic acceleration $[a]$ of ventilation stack.

Limit seismic acceleration $[a]$ may be considered as random value.

Sources of spread of limit seismic acceleration could be divided into two groups:

- random nature of the factors determining $[a]$;
- uncertainty of techniques of an estimation $[a]$.

To the first group belong properties of construction materials and soils, tolerances of geometric dimensions etc. Let's specify these factors as the factors of a type R.

Uncertainties of existing analytical techniques also cause deviations of estimated value of $[a]$ from its true value. Formally these deviations seem as random spread and give the contribution to the total spread of $[a]$. Let's specify the factors connected to this component of spread as the factors of a type U. Basic difference of these factors from the factors of a type R is that by development of more correct analytical techniques spread of $[a]$ connected to them could be reduced, while spread connected with factors of a type R could not be reduced.

Main factors of spread of limit seismic acceleration [*a*], which were studied in present work, are shown in the Table 1

Table 1. Main factors of spread of limiting seismic acceleration [*a*]

i	Factor	Type
1	Inexactness in formulation of limit state of construction	*U*
2	Variability of strength parameters of materials (concrete, reinforcing)	*R*
3	Inexactness in estimation of energy absorption at oscillations	*R, U*
4	Variability and inexactness of parameters of soil-structure interaction	*R, U*

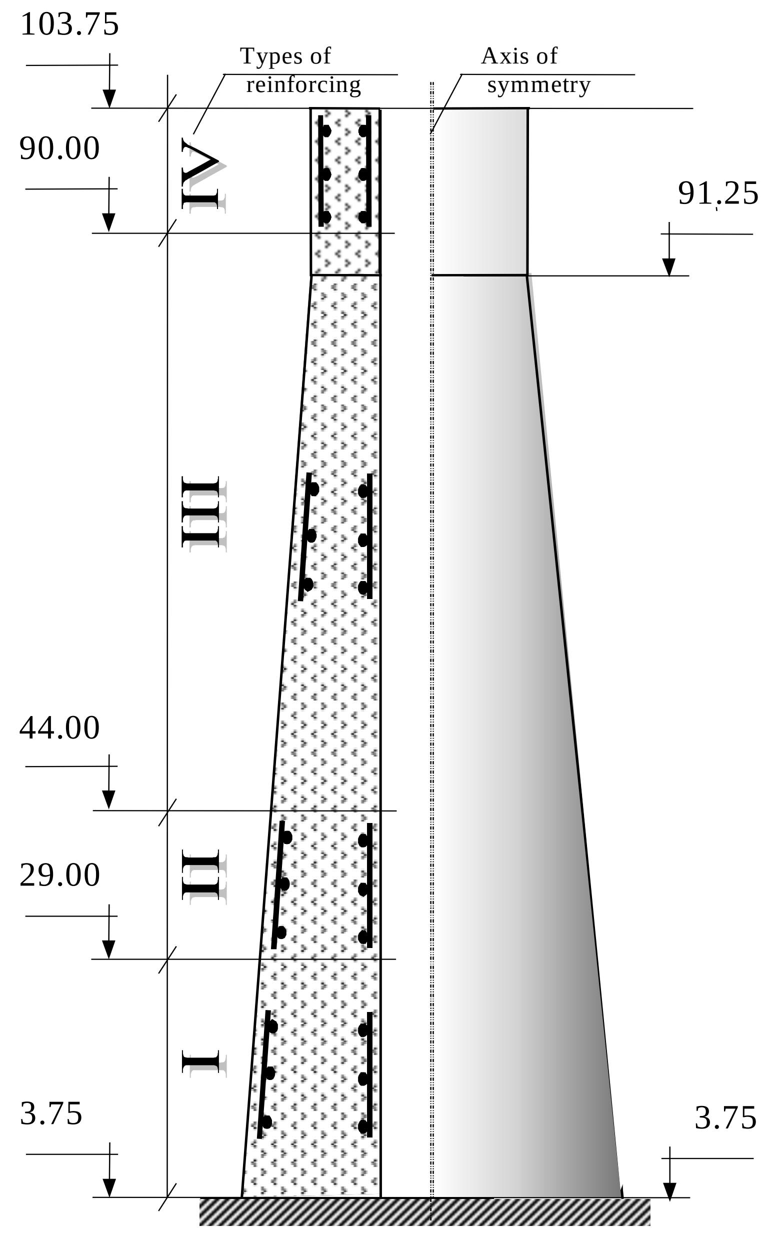

Fig.1. Ventilation stack of nuclear power plant (dimension in meters)

In present work 'scaling method' was used for calculating of fragility curves $P_0(a,Q)$. Base principles of this method are presented in the works [2, 3] developed by *R.P.Kennedy et al.* By using of this method fragility could be estimated on the base of deterministic estimation of limit seismic acceleration considering statistical properties of factors, which define its spread.

In accordance with scaling method random value [*a*] is assumed to be distributed lognormally (Fig.2). On the curve of this distribution there are two reference points: $m_{[a]}$ and [a_0]. Value $m_{[a]}$ corresponds to median of distribution. In scaling method this value is used as approximate estimation of the center of distribution. Value [a_0] corresponds to deterministic estimation of limit seismic acceleration [a_0]. In scaling method design value of [*a*], obtained by using existing codes is used as [a_0]. Because of margins, introduced in deterministic analysis by codes and by designers, value [a_0] as a rule lies on the curve on the left from $m_{[a]}$.

To obtain fragility function, it is necessary to define median $m_{[a]}$ and logarithmic standard deviation β of distribution. It is convenient to use separately parts of β, connected with factors of type *R*, β_R, and factors of β, connected with factors of type *U*, β_U:

$$\beta = \sqrt{\beta_R^2 + \beta_U^2} \ ,$$

If values of $m_{[a]}$, β_R, and β_U are known, fragility curve for some value of confidence probability *Q* can be defined as:

$$P_0(a,Q) = \Phi\left[\frac{\ln(a/m_{[a]}) + \beta_U \Phi^{-1}(Q)}{\beta_R}\right], \quad (1)$$

where

Φ - integral function of normal distribution,
Φ^{-1} - function, inverse to Φ.

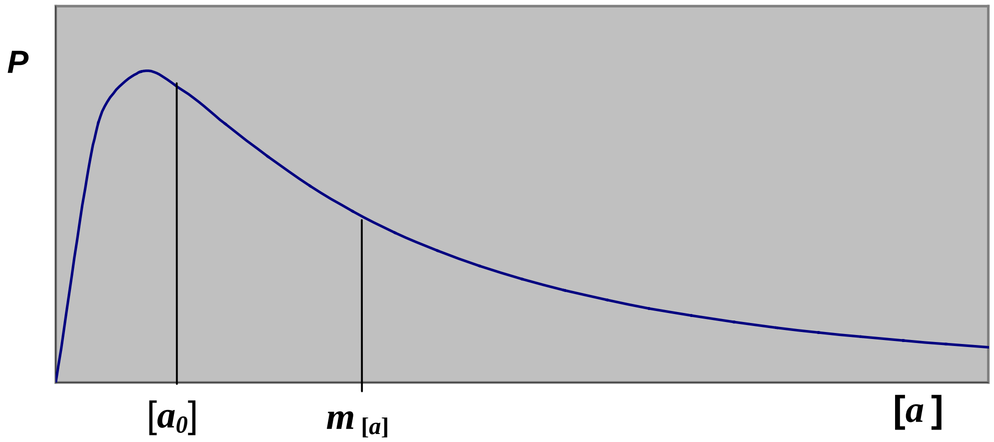

Fig.2. Distribution of limit seismic acceleration

In accordance with concept of scaling method median $m_{[a]}$ could be defined on the base of deterministic estimation of limit seismic acceleration $[a_0]$,:

$$m_{[a]} = F_1 \cdot F_2 \cdot \ldots \cdot F_n \cdot [a_0], \tag{2}$$

where F_i - coefficients, connected with different factors of spread of $[a]$, which reflect design margins.

To define coefficients F_i it is necessary to obtain by engineering analysis dependence of limit seismic acceleration against value of i-th factor of spread $[a](x_i)$.

If such dependence is obtained coefficients F_i could be defined as:

$$F_i = \frac{[a](m_i)}{[a](x_i)}, \tag{3}$$

where m_i - value of i-th factor, which corresponds to median of its distribution
xi - value of i-th factor, which was accepted in deterministic analysis.

Logarithmic standard deviation β_R, and β_U could be defined by generalization of logarithmically standard deviations connected corresponding factors of spread:

$$\beta_R = \sqrt{\sum \beta_{Ri}^2}\ , \ \beta_U = \sqrt{\sum \beta_{Ui}^2}\ . \tag{4}$$

Each i-th logarithmic standard deviation could be defined as:

$$\beta_i = \frac{1}{q_i} \ln\left\{\frac{[a](m_i)}{[a](x_i)}\right\}, \tag{5}$$

where q_i - number of standards between median m_i and design value x_i of i-th factor.

FRAGILITY ANALYSIS

Deterministic seismic analysis of an ventilation stack was fulfilled by using unit spectra of seismic excitation (function of dynamic coefficient) shown in Fig.3. The frequencies and forms of a natural oscillations of the stack were determined in elastic approach by modal analysis. Soil base was approximated by elastic hemispace. Forms of natural oscillations are presented in Table 2. First form of natural oscillations provides the main contribution in overall dynamic stresses.

Table 2. Forms of natural oscillations of ventilation stack

№ of form	Period *T*, s	Type of oscillations
1	1.458	bending by 1st beam form
2	0,409	bending by 2nd beam form
3	0,285	vertical oscillations due to compliance of soil base
4	0,182	bending by 3rd beam form
5	0,130	bending by 4th beam form

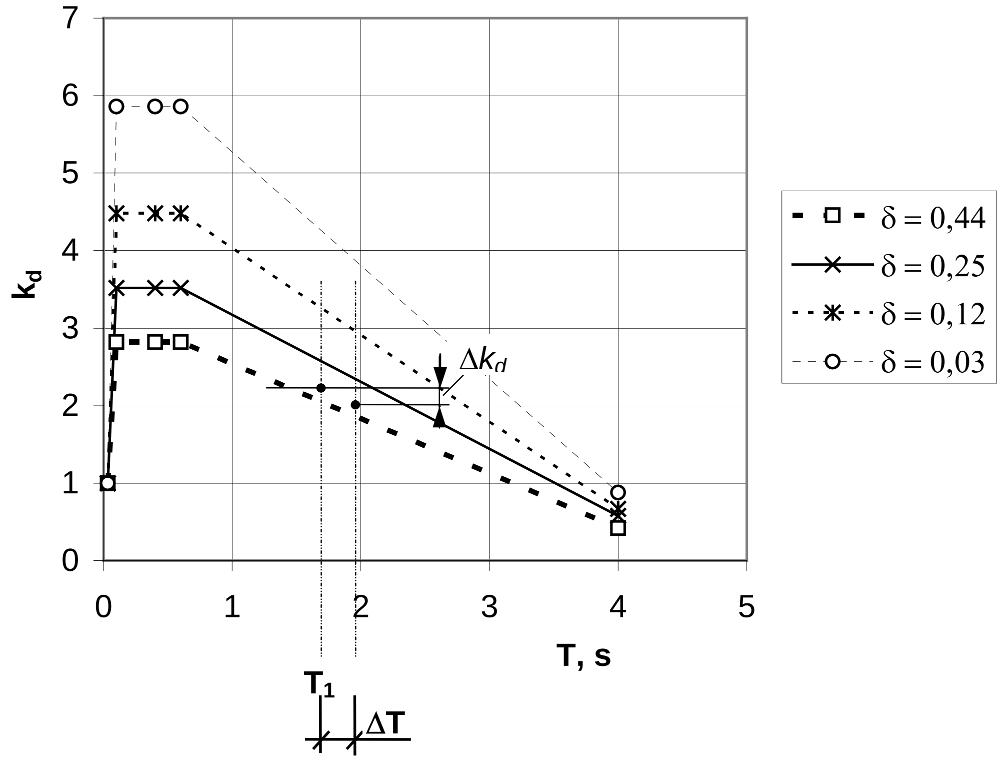

Fig.3. Dependence of dynamic coefficient on period of natural oscillations for different values of logarithmic decrement δ

For the stress analysis finite element method was used. For the unit seismic acceleration $\overline{a} = 1$ m/s^2 margins in different levels of the stack were estimated with account of deadweight. Margins of the section at the level of 77 m appeare the least. On the base of these margins was determined deterministic value of limit seismic acceleration:

$$[a_0] = 2{,}02 \text{ m/s}^2. \quad (6)$$

Using this value fragility analysis of the stack was fulfilled by scaling method with obtaining values F_i , β_i for each factor listed in table 1.

As an example of definition of values F_i , β_i for the factors of a type *R* let us consider factor $i = 2$ - variability of strength parameters of concrete. In present work yield stress σ_T was used as strength parameter for concrete. This parameter may be considered as normally distributed random value (Fig.4).

In accordance with Russian civil engineering standards stress analysis of structures should be fulfilled by limit state method using values of characteristic resistance R_{bn} and design resistance R_b. These values are shifted on 1,64 and 3 standard deviations *s* relatively median of distribution m_R correspondingly.

For concrete used in ventilation stack values of R_{bn} and R_b for given class of concrete are defined in code [5]: $R_{bn} = 18{,}5$ MPa, $R_b = 14{,}5$ MPa.

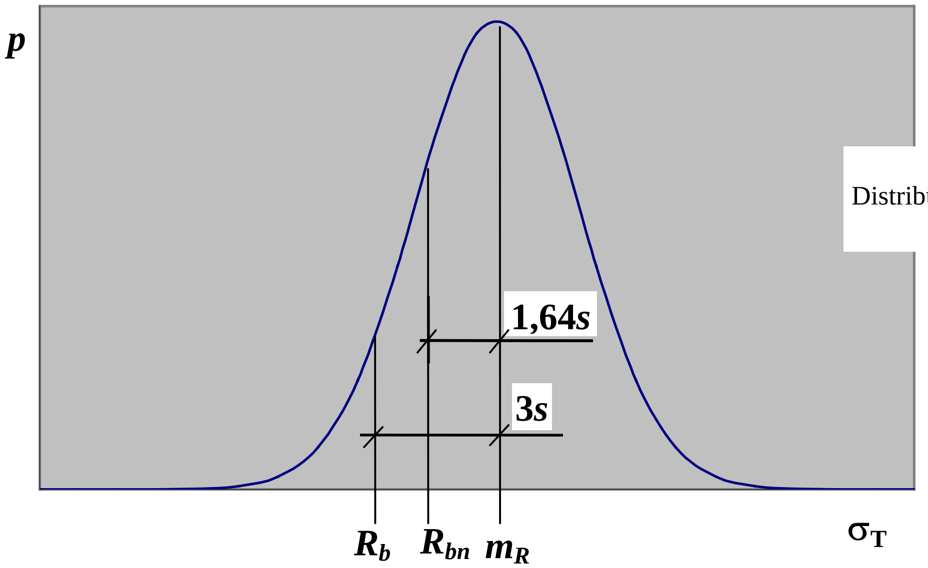

Fig.4.
Distribution of strength of concrete

In above considered analysis margins were defined on the base of characteristic resistance $R_{bn.}$ So, by using (3) and (5) coefficient F_2 and partial logarithmic standard deviation β_{R2} may be obtained as:

$$F_2 = \frac{m_R}{R_{bn}} = 1{,}26\,, \tag{7}$$

$$\beta_{R2} = \frac{1}{1{,}64} \ln F_{K2} = 0{,}14. \tag{8}$$

Because considered factor belongs to type *R*,

$$\beta_{U2} = 0\,. \tag{9}$$

Other example relates to the factor $i = 4$ (variability and inexactness of parameters of soil-structure interaction) which belongs to both types - *R* and *U*.

Coefficient F_4 connected with this factor could be assumed as margins introduced in value [*a*] by simplifications in soil-structure interaction models adopted in deterministic analysis. Simplified models do not take into account wave nature of interaction and different wave effects, including reemission of energy back into soil, which reduce dynamic reaction of structure.

Deterministic value of limit seismic acceleration (6) was defined considering that foundation of stack may be modeled by rigid stamp. Because of rigidity and compactness of real foundation in the case of ventilation stack such model could be assumed as quite adequate.

Therefore margin coefficient could be adopted as:

$$F_4 = 1\,. \tag{10}$$

The spread of limit seismic acceleration [*a*], connected with the factor $i = 4$, is defined by randomness of parameters of soil base and also by uncertainties of existing techniques of definition of its dynamic reaction. First contribution of this spread could be taken into account by coefficient β_{R4}, second – by coefficient β_{U4}.

Variation in properties of the soil base lead to variation in period of natural oscillations ΔT. The latter cause variation in dynamic coefficient k_d (Fig.3). Deterministic value of limit seismic acceleration (6) was obtained considering elastic work of materials. At such approach variation of [*a*] could be assumed as inversely proportional to variation of k_d. If the latter corresponds to one standard deviation of considered property of soil base coefficients β_{R4}, β_{U4} could be obtained by (5), using $q_4 = 1$:

$$\beta_4 = \ln\left(\frac{k_d + \Delta k_d}{k_d}\right) = \ln\left(1 + \frac{\Delta k_d}{k_d}\right) \quad (11)$$

In the case of rigid stamp model of foundation coefficient of rotational rigidity of the soil base K_ϕ became the main parameter, which define T.

This coefficient could be presented as product of modulus of elasticity of soil E and some function of geometric parameters of foundation Θ:

$$K_\varphi = E \cdot \Theta . \quad (12)$$

Modulus of elasticity E is a physically random value. Its variations define R-part of spread of value [a], connected with factor $i = 4$. Relative standard deviation of [a] in accordance with (11) could be expressed as:

$$\beta_{R4} = \ln\left(1 + \frac{\Delta_R k_d}{k_d}\right), \quad (13)$$

where $\Delta_R k_d$ - variation of dynamic coefficient due to random variation of modulus of elasticity E of soil.

Function Θ for round foundation on natural soils could be calculated in accordance with recommendations of different authors [4]. Corresponding results could vary significantly. These variations define U-part of spread of value [a], connected with factor $i = 4$. Relative standard deviation of [a] in accordance with (11) could be expressed as:

$$\beta_{U4} = \ln\left(1 + \frac{\Delta_U k_d}{k_d}\right), \quad (14)$$

where $\Delta_U k_d$ - variation of dynamic coefficient due to uncertainty in estimation of Θ.

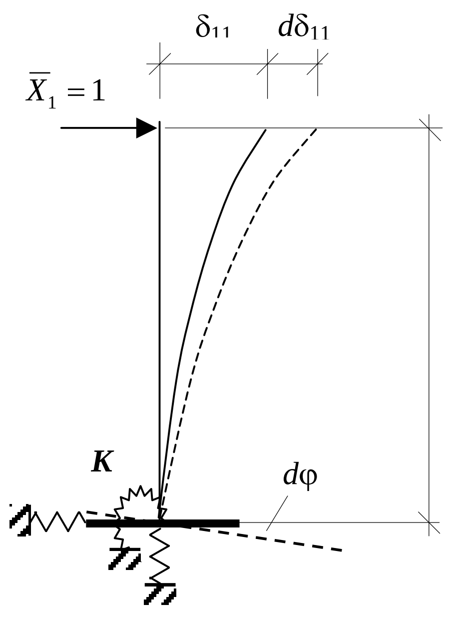

Fig.5. Beam oscillations of ventilation stack

For obtaining of values β_{R4}, β_{U4} it is necessary to find connection between variation of coefficient of rotational rigidity of the soil base ΔK_ϕ and variation of dynamic coefficient Δk_d. Because 1st beam form of oscillations provides main contribution in dynamic reaction of the stack, such connection could be obtained, considering stack as a beam (Fig.5). Period of first form of oscillations is connected with unit displacement of the beam δ_{11}. Variation of K_ϕ by some increment ΔK_ϕ causes change in unit displacement δ_{11} by increment $d\delta_{11}$. The latter is connected with additional rotation of foundation by angle $d\varphi$.

Unit displacement δ_{11} consist of two parts:

$$\delta_{11} = \delta_0 + \delta_\varphi ,$$

where the first one reflects the bending of the shaft and the second one reflects rotation of the foundation.

If neglecting bending of rigid reinforced concrete shaft for approximate engineering estimations, variation of first period of natural oscillation could be expressed as:

$$\Delta T = T\left(\sqrt{1 + \frac{\Delta K\varphi}{K\varphi}} - 1\right) \quad (15)$$

If value ΔT, is obtained corresponding variation of dynamic coefficient $\Delta_{R,U} k_d$ could be found by curves of Fig.3.

To obtain $\Delta_R k_d$ it is necessary to substitute in (15) variation ΔK_ϕ connected with random variation of E. Because of linear dependence (12) between K_ϕ and E expression (15) in this case could be written as:

$$\Delta T_R = T(\sqrt{1+\frac{\Delta E}{E}} - 1) \;. \tag{16}$$

or in the case, when ΔE corresponds to one standard deviation of modulus of elasticity

$$\Delta T_R = T(\sqrt{1+\zeta_E} - 1) \;, \tag{17}$$

where ζ_E - coefficient of variation of modulus of elasticity of soil.

In accordance with data presented in [5], for clay and sand coefficient of variation of modulus of elasticity could be assumed as $\zeta_E = 0{,}4$. Taking into account this value, one could obtain: $\Delta T_R = 0{,}18T = 0{,}26$ s, $\Delta_R k_d = 0{,}2$ and then by using (13):

$$\beta_{R4} = 0{,}07 \;.$$

Uncertainty in estimation of Θ could be estimated by calculation of value Θ using formulas of different authors and obtaining mathematical expectation (median), and standard deviation of the set of results. On the base of this data coefficient of variation ζ_Θ could be defined.

Analogously to (17) variation of first period of natural oscillation due to uncertainty of estimation of soil base rigidity could be expressed as:

$$\Delta T_U = T(\sqrt{1+\xi_\Theta} - 1) \;. \tag{18}$$

As a result of above mentioned analysis was obtained value of $\zeta_\Theta = 0{,}3$. Taking into account this value, one could define: $\Delta T_U = 0{,}14T = 0{,}20$ s, $\Delta_U k_d = 0{,}125$ and then by using (14):

$$\beta_{U4} = 0{,}05 \;.$$

RESULTS OF ANALYSIS

Resulting values of coefficients F_i and β_i for all four factors *i* defining spread of limit seismic acceleration [*a*] of ventilation stackare shown in table 3.

Table 3. Parameters of spread of limit seismic acceleration for ventilation stack

Factor ***i***	Coefficients F_i	Logarithmic standard deviations	
		β_{Ri}	β_{Ui}
1	2,49	0	0,17
2	1,26	0,14	0
3	1,33	0,11	0,22
4	1	0,07	0,05

Taking into account values F_i and deterministic value of limit seismic acceleration (6) by using expression (2) estimation of median of value [*a*] could be obtained:

$$m_{[a]} = 9{,}5 \text{ м/s}^2. \tag{19}$$

Taking into account values of β_{Ri} , β_{Ui} by using (4) common logarithmic standard deviations of value [*a*] for factors of *R*-type and *U*-type could be obtained:

$$\beta_R = 0{,}19\,;\ \beta_U = 0{,}29\,. \tag{20}$$

Generalized logarithmic standard deviations of [a] will be equal to:

$$\beta = 0{,}34\,. \tag{20}$$

Using obtained values fragility curves could be plotted for different levels of confidence probability Q (Fig.6).

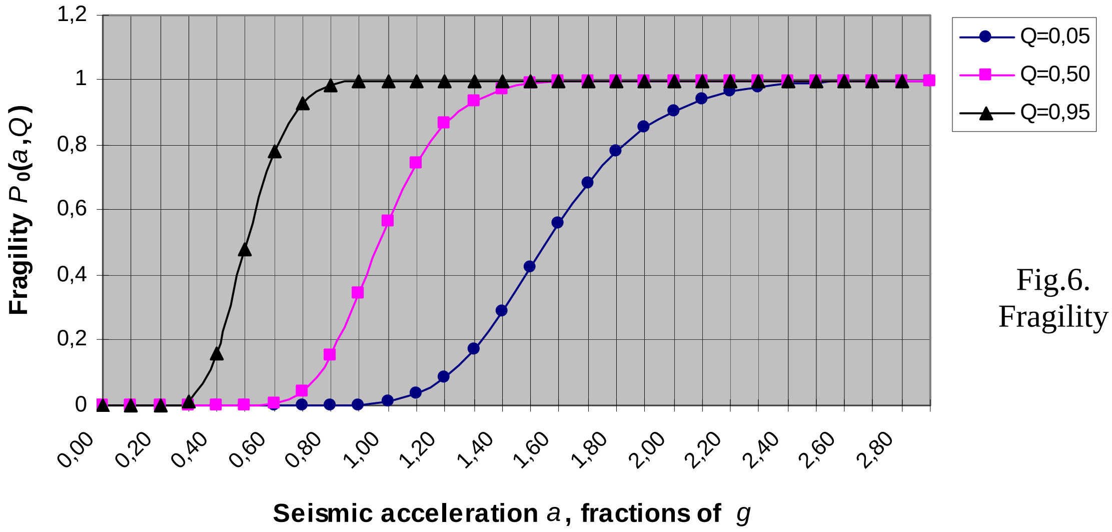

Fig.6. Fragility

ACKNOWLEDGEMENTS

Authors would like to express their gratitude to directorate of Scientific and Engineering Centre for Nuclear and Radiation Safety of the Federal Nuclear and Radiation Safety Authority of Russia and especially to Deputy Director Mrs. I.V.Kaliberda for moral and organization support of the work.

REFERENCES

1. INTERNATIONAL ATOMIC ENERGY AGENCY, IAEA-TECDOC-724: Probabilistic safety assessment for seismic events, IAEA, October, 1993.
2. Kennedy R.P. et al. Probabilistic Seismic Safety Study of an Existing Nuclear Power Plant. “Nuclear Engineering and Design”, v.58 (1980), p.p. 315-338.
3. Kennedy R.P., Ravindra M.K. Seismic Fragilities for Nuclear Power Plant Risk Studies. “Nuclear Engineering and Design”, v.79 (1984), p.p.47-68.
4. Sargsyan A.E., Bednyakov V.G. Seismic Resistance of Structures and Soil Bases of Nuclear Plants. Review information. M., Informenergo, 1990. (in Russian).
5. Klyamcionok P.L. Indirect methods of definition of parameters of properties of soils. Leningrad, Stroyizdat, 1987. (in Russian).