1

Paper No. 538. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

PORCE II - DYNAMIC RESPONSE OF THE DAM AND REINFORCEMENT FILL

L.F.RESTREPO V.[1] and M.R.VILLARRAGA H.[2]

Geotechnical Department, Integral S.A., Carrera 46 N°52-36, Edif. Vicente Uribe Rendón, Medellín, Antioquia, Colombia.

ABSTRACT

Porce II Hydroelectric Project, owned by Empresas Públicas de Medellín, now under construction, is located 120 km north-east from the city of Medellín in the Department of Antioquia, in an area of intermediate seismicity.

Main civil works comprise a gravity concrete dam to be constructed by roller compacted concrete technology (RCC), with a maximum height of 123 m and a concrete volume of 1'300.000 m^3. The dam is located in a narrow and unsymmetrical valley, with adequate rock foundation in the left abutment, while in the upper part of the right abutment, the natural terrain is flat and adequate rock foundation is deep, making it necessary to place a reinforcement fill in this abutment with a maximum height of 40 m and a total volume close to the main concrete dam.

This paper includes the results of the analysis carried out to define the dynamic response of the dam, reinforcement fill and the interaction between them. In addition, it presents the main aspects of the structures, as related to the optimization of the design to reduce seismic risk.

KEYWORDS

Porce II, Dynamic Analyses, RCC, Reinforcement Fill, Probabilistic Approach.

INTRODUCTION

Porce II Hydroelectric Project, owned by Las Empresas Públicas de Medellín, is located about 120 km northeast of Medellín, Department of Antioquia, Colombia.

Project construction began on January, 1995, and it is expected to begin commercial operation by 1999. Basically, the project includes a 118 m high Roller Compacted Concrete (RCC) gravity dam, combined with

[1] Junior engineering, Integral S.A.; Facultad de Ingeniería, Universidad de Medellín.
[2] Senior engineering, Integral S.A.; Facultad de Minas, Universidad Nacional.

an earth-rock embankment placed at the right abutment of the main dam. Reservoir capacity will be 120'000.000 m^3. Headrace tunnel will be 5.9 km long; the power house will be built inside an underground chamber 91,7 m long, 21,3 m wide and 43 m high and it will house 3 Francis turbines for a total capacity of 392 MW.

Integral S.A, a consulting firm from Medellín (Colombia) with 40 years of experience in hydroelectric developments in Colombia and in other South American countries, was in charge of the design of the project for Empresas Públicas de Medellín. Design works were accomplished between 1989 and 1995. This paper presents a synthesis of the studies carried out to define the dynamic response of the RCC and earth-rock fill dams and the interaction between them, and describes the measures needed to guarantee an earthquake-resistant structure (Integral S.A., 1995).

DESCRIPTION OF THE DAM

Figure 1 shows a layout of the general dam configuration for Porce II. The main dam will have a height of 118 m above minimum foundation level; the crest length will be 455 m and 9 m wide. The dam will have 1'415.000 m^3 of concrete, with 1'300.000 m^3 RCC, 68.000 m^3 facing concrete, 400 m^3 leveling concrete and 46.000 m^3 conventional concrete for the spillway, which will be incorporated in the central portion of the gravity dam. At the right abutment, there will be a 1'620.000 m^3 earth-rock reinforcement embankment, 45 m high.

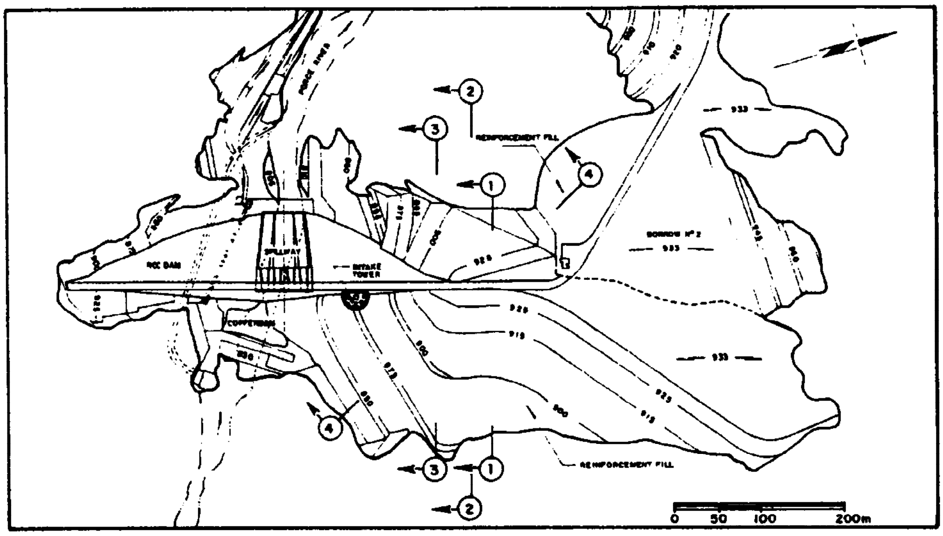

Fig. 1. General dam configuration for Porce II

Porce II dam is located on a narrow unsymmetrical valley which corresponds to the contact zone between hornfels-type metamorphic rocks (outcrop on the left slope) and quartzdiorite rocks which generate thick residual soils. The slopes are covered locally by consolidated and weathered mudflows.

Good rock conditions encountered on the left abutment and on the river bed are suitable to support the RCC dam, whilst the right abutment, where residual soils coming from quartzdiorite are found, sound rock was only detected very deep inside the ground; therefore, an adequate foundation for a concrete gravity dam was not guaranteed at this side. Because of this, it was necessary to design a reinforcement earth-rock fill dam to supplement the closure of Porce river valley.

Figure 2 shows two typical sections of the gravity dam. Figure 3 presents typical sections of the reinforcement fill dam, which will have an internal embankment up to 45 m high, composed by compacted sandy silt (ML) or silty sand (SM), product of quartzdiorite weathering, which will be in direct contact with the RCC dam. This fine soil core will be protected on both sides by unweathered hornfels rock shells, which will guarantee the external slope stability. The earth-rock fill slopes will be 3.5 Horizontal to 1 Vertical.

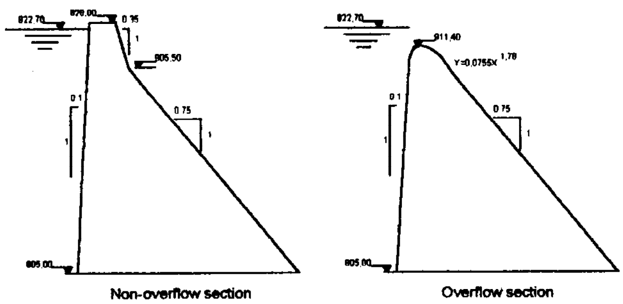

Fig. 2. Typical sections of the gravity dams

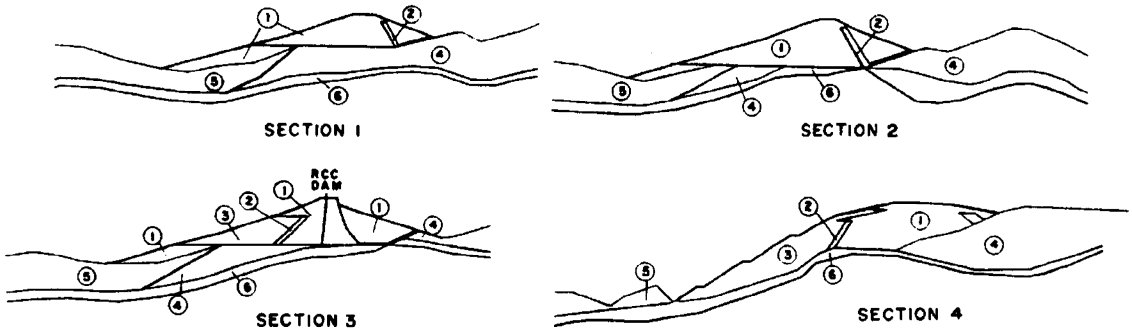

Fig. 3. Typical sections of the reinforcement fill dam

The reinforcement fill will stand over a thick sandy silt and silty sand layer from weathering of quartzdiorite and old deposits from the upstream sector. This layer is up to 20 m thick and presents variable strength and deformation characteristics, which improve at depth. The fill zone around the gravity dam will be founded on slightly weathered hornfels rock.

SEISMIC CONDITIONS

The seismicity to which the project works are exposed could be divided in a deep source, associated to the Benioff Zone of the Nazca plate subducting South American plate 80 km below the project site, and a shallow one, associated to potentially active faults such as the remarkable Espíritu Santo Fault, located 50 km from the dam site and with signs of recent activity.

In accordance with the seismic hazard study, the maximum horizontal accelerations expected are 0.15, 0.19 and 0.22 g, for 200, 500 and 1000 years of recurrence respectively.

Normalized acceleration response spectrum was used for the design and was obtained on the basis of those proposed by Joyner & Boore (1988). Real and adjusted to the proposed spectral from synthetic accelerograms were generated in order to use them in the structure analysis.

For the reinforcement fill, it was used also the Opogadó earthquake record (1992), obtained by ISAGEN in an accelerograph located in Medellín.

EVALUATION OF THE GRAVITY DAM DYNAMIC RESPONSE

Methodology

In order to establish the dam risk level to dynamic loads, a probabilistic analysis was carried out taking into account 12 different accelerograms obtained from real and synthetic generated earthquakes (Integral S.A., 1991). Four load cases were analyzed for each generated earthquake, including empty reservoir, full reservoir without uplift and full reservoir plus uplift, for both, the maximum non-overflow and spillway sections.

Dynamic properties of the materials

At the beginnig of the study, dynamic properties for concrete and foundation rock were assumed, based on available information from projects under similar conditions. Later on, these properties were confirmed or corrected, from actual results from the laboratory test program. Table 1 shows a summary of the properties used in the analysis.

Dynamic Analysis

Dynamic analysis was carried out by using the EAGD-84 computer program (Fenves and Chopra, 1984), which considers a two dimensional structure under plain strain conditions considering water compressibility, rock flexibility, sediments effect and hidrodynamic wave dissipation. As normally taking into account, uplift was considered unchanged during the seismic event (USBR, 1976).

Probabilistic Analysis

In experimental stress analysis, static stresses are supposed to fit a normal o Gaussian frequency distribution (Durelli *et al.*, 1958). This is not clear for the distribution of dynamic stresses, and a large number of cases would be required to perform the statistical analysis.

Table 1. Material Properties

PROPERTY	CONCRETE	ROCK
Dynamic Modulus (GPa)	40	50
Poisson's ratio	0,15	0,33 (2)
Mass density (kN/m³)	24	24
Hysteretic Damping (%)	10	10
Dynamic Tensile Strenght (MPa) (1)		
Horizontal	2,7	
Vertical	2,2	

(1) 365 days
(2) EAGD-84 does not accept any other value

The basic assumptions to carry out the probabilistic analysis of the maximum dynamic stresses were: the maximum tensile stresses developed for each synthetic earthquake and for any of the load cases considered have the same probability of occurrence and the same maximum tensile stresses data are normally distributed (Walpole and Raymond, 1968).

Due to the reduced sample size and the unknown standard deviation of the universe, a t-Student distribution was used to calculate the confidence limits for the mean value. The confidence intervals considered were 90, 95 and 98%, which correspond to 0.05, 0.025 and 0.01 areas (α) respectively .

Evaluation of results

In the statistical analysis a mean value of 1.78 MPa was obtained with a standard deviation of 0.43 MPa for the maximum tensile stress distribution in the most critical condition, considering the full reservoir load case. Table 2 shows confidence intervals of dynamic tensile stresses for 95% significance level, assuming they fit a normal distribution.

Table 2. Confidence intervals for 95% mean maximum dynamic tensile stresses (MPa)

Case	Non-overflow section			Overflow section		
(1)	H.	H.+V.	H.+V(6)	H.	H.+V.	H.+V(5)
Empty reservoir	1,00	1,14	1,00	0,54	0,62	0,54
(2)	1,28	1,42	1,26	0,76	0,82	0,76
Full reservoir	1,18	0,90	1,24	0,98	0,81	1,12
w/o uplift(3)	1,68	1,34	1,78	1,48	1,17	1,64
Full reservoir	1,44	1,15	1,51	1,26	1,06	1,37
with uplift(4)	1,94	1,63	2,05	1,70	1,46	1,89

(1) Twelve earthquakes were used for each load case.
(2) Stresses at the top of downstream face.
(3) Stress at heel of the dam.

In addition, a probabilistic analysis was carried out for maximum dynamic tensile stresses generated at critical points of the dam, for every load combination, so as the probabilistic analysis for the whole bulk of data from cases corresponding to the full reservoir condition for the two sections.

Table 3 summarizes the obtained results of the full reservoir condition analysis. A maximum excedence probability of 11% resulted for the vertical and horizontal earthquake for the maximum non-overflow section. A 5% maximum excedence probability was obtained for the maximum overflow section.

Table 3. Probability of exceeding the dynamic strength of concrete

Earthquake	Maximum non-overflow section			Maximum overflow section		
	P	Mean	S.D.	P	Mean	S.D.
H.	6%	1,60	0,40	1,4%	1,40	0,37
H.+V.	11%	1,30	0,40	5%	1,20	0,31
H.+V.(Inv)	0,5%	1,70	0,43	0,05%	1,50	0,41
All components	4,9%	1,50	0,43	1,7%	1,35	0,40

EVALUATION OF REINFORCEMENT FILL DYNAMIC RESPONSE

Methodology

This analysis was carried out in order to evaluate the reinforcement fill response under different load conditions, specially those due to earthquakes, when it reacts as an independent structure and in the case of interaction with the gravity dam.

The methodology applied to the study is based on that proposed by Seed, Idriss and Lee (1970) in which the static and dynamic stress conditions were evaluated on the basis of finite element bidimensional models. Static parameters and properties, were defined from laboratory test results made on undisturbed and representative samples. Empirical and semi-empirical correlations were used for dynamic properties of the fill materials coming from a number of dynamic tests results obtained from materials with similar characteristics used for the construction of several dams in Antioquia (Villegas *et al.*, 1987; Integral S.A., 1991).

The previous static stresses analysis was obtained for different conditions that could be present during the useful life of the project (construction, operation, reservoir fill, etc.) for every section. Finite element software was used to simulate the nonlinear behavior of the soil and the construction process (Duncan *et al.*, 1980).

Taking in account the static stress states, program LUSH2 (Lysmer *et al.*, 1974) was used to calculate the earthquake-induced stress increase. Based on the induced increments and the available laboratory results, the deformations and interstitial pressure increments were estimated. These results were used to review the stability of the fill in post-seismic conditions, considered as specially critical.

Dynamic Properties of Materials

The shear modulus was expressed as function of the stress, the deformation level and the semi-empirical parameter K_2, in accordance with the expression:

$$G = 1000^* K_2 \, \sigma \, f(\gamma) \qquad \text{p.s.f.} \qquad (2)$$

where: K_2 : Adimensional parameter
σ : Principal Effective Geostatic Strain
$f(\gamma)$: Modulus change with the deformation level

Table 4. K_2

Material	Description	K_2
1	Silty soils fill	60
2	Filter	100
3	Rock fill	100
4	Fundation residual	80
5	Mudflows	80
6	Wheatered rock	100

The curves proposed by Dobry and Vucetic (1991), were used to define the shear modulus ratio and the critical damping ratio and their variation with the deformation level for the silty soils of the embankment, and for the foundation residual soils, in accordance with the plasticity index. Curves proposed by Seed *et al* (1970) were used with the weathered rock and filters. On the other hand, curves obtained from the El Infiernillo dam in Mexico (Villarraga, 1987) were used on the rockfill.

The dynamic pore pressure generation curves were defined from the cyclic test results over soils from other dams constructed with similar materials, such as Punchiná, Jaguas, Troneras, Miraflores and Riogrande II (Villegas *et al.*, 1987). Figure 4 shows results from variation between stress cyclic ratio ($\sigma 2/2\sigma 3$) and the number of cycles needed in order to reach the maximum water pore pressure.

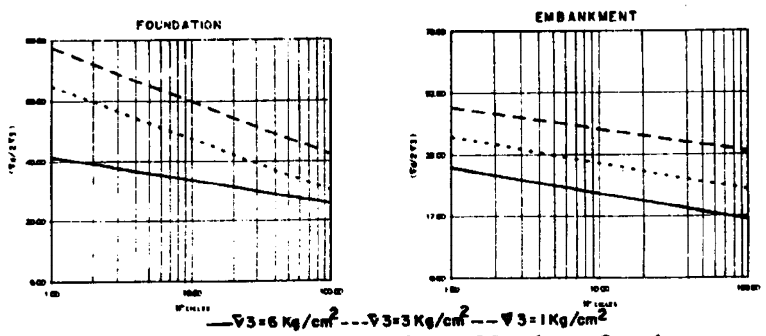

Fig. 4. Cyclic stress ratio vs. Number of cycles

Dynamic Analysis

The four sections mentioned above were analyzed on the operation (full reservoir) and end of construction conditions, while two synthetic earthquakes fitted to the designs response spectrum, were used as excitation sources due to their broad frequency. The earthquakes were scaled to 0.22g and 0.19g accelerations, which correspond to 1000 and 500 years recurrence interval, and that could be used as the M.C.E. (Maximum Credible Earthquake) and the B.O.E. (Basic Operation Earthquake) respectively. In order to evaluate the frequency band effect, the Opogadó earthquake was scaled to 0.22g as the maximum acceleration.

For sections 1, 2 and 4 , the fundamental structure periods obtained were 0.8 and 1.0 sec., that correspond to the end of construction and the dam operation conditions respectively. This increase in period is due to decrease of rigidity of materials, associated to reduction of effective stress as a consequence of the reservoir fill. The vibration characteristics of these sections are similar, although they present different heights of the embankment.

Section 3 response is controlled by the gravity dam, resulting in lower fundamental periods if compared with the other sections, but higher than those calculated for the RCC dam independently. Figure 5 shows the comparisons obtained from the results of the acceleration responses spectra from the top of sections 1 and 3. The difference in response for these sections suggests the possibility of cracks and fissures perpendicular to the dam axis, so this confirms the need of protecting the fill contacts with a concrete dam, with thick filters in order to prevent particle dragging and reduce the probable tubification risk to a minimum.

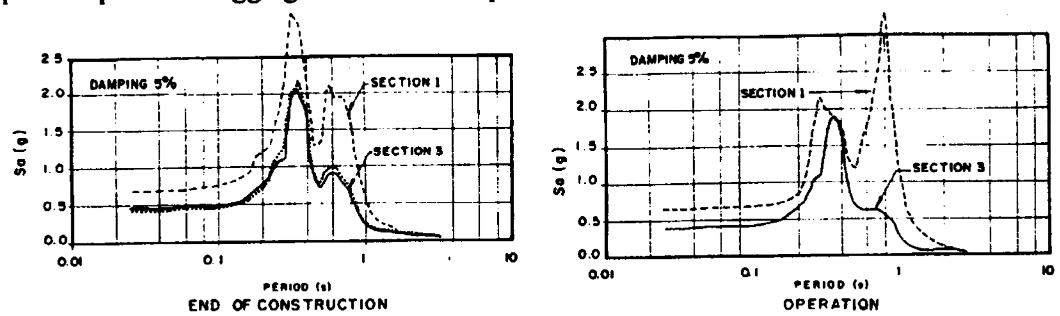

Fig. 5. Comparison of acceleration response spectra

The earthquake frequency band influence on the structures response, can be seen from results obtained from the synthetic earthquake with broad band response and the narrow band Opogadó earthquake.

As a consequence of the earthquake, water pore pressures are generated and the structure stability reduced. The dynamic water pore pressure relations ($r_{ud,}$ defined as the induced pore pressure relation over the effective vertical strain) reached values greater than 0.7 and slightly lower for the Opogadó earthquake. Interstitial pore pressures are lower in section 3 due to the presence of the gravity dam which constrains the fill deformations.

CONCLUSIONS

Considering that the exceptional earthquake selected has a return period of 1000 years or more, the analysis provides an idea of the exceedance probability of the specified dynamic tensile strength of concrete within the considered return period.

On the average, the dynamic tensile strenght of concrete has a probability of exceedance of 4% at the heel of the maximum non-overflow section, for an exceptional earthquake (1000 years or more return period). Similarly, the probability of exceeding the dynamic tensile strength of concrete at the heel of the over flow section would be 2% on the average.

It should be pointed out that dynamic stresses are highly variable and several cycles of high tensile stresses may be required for a crack develop. In addition, the probability of exceeding the dynamic tensile strength is higher at the heel of the maximum non-overflow section in comparison with the maximum overflow section. However the former is shorter than the latter.

Due to the differences of rigidity between the RCC dam and the reinforcement fill, there are different vibration characteristics among the gravity dam, the reinforcement fill contact zone and the embankment itself. This condition could result in the development of cracks perpendicular to the dam axis; hence it is necessary to protect the fill-dam contact materials with thick filters.

The reinforcement fill can be considered as secure in the event of a 1000 year recurrence earthquake, specially in that zone near the gravity dam, due to the small width of residual soil in the foundation and the rockfill slope. Towards upstream (section 1 and 2) there could be development of failures that do not involve the global stability, due to the great volume and widens of the crest in this sector. For 500 year earthquakes the fill stability is guaranteed, placing drainage layers and constructing the proposed fill on the embankment toe, which will act as a counterweight.

ACKNOWLEDGMENTS

The authors thanks Empresas Públicas de Medellín for authorizing the publication of this paper, to Integral S.A. for its constant encouragement. Thanks to Engineers Fabio Villegas, Eduardo Lopera and Camilo Tejada for their observations, comments and revisions; also to those who collaborated on the Spanish-English translation.

REFERENCES

Duncan J., Wong K., Osawa Y. (1980), FEADAM: A Computer Program for Finite Element Analysis of Dams. University of California. Berkeley.

Fenves, G., Chopra, A.K. (1984) EAGD-84 A Computer Program for Earthquake Analysis of Concrete Gravity Dams, Report N° UCB/EERC-81/11. University of California. Berkeley, California.

Integral S.A. (1991). Revisión del Estudio de Riesgo Sísmico, Proyecto Hidroeléctrico Porce II - Estudios e Investigaciones - Sismología, Documento 7108-04 Rev.0, Empresas Públicas de Medellín. Medellín.

Integral S.A. (1995 a). Porce II Dam - Probabilistic Dynamic Stress Analysis. EEPPMM. Medellín.

Integral S.A. (1995 b). Proyecto Porce II - Análisis de Estabilidad Estático y Dinámico del Lleno de Refuerzo de la Margen Derecha, Informe Final. Empresas Públicas de Medellín. Medellín.

Joyner W., Boore D. (1988). Recent Advances in Ground Motion Evaluation, EESD II. ASCE. Park City.

Lysmer J., et al (1974). LUSH-2, A Computer Program for Complex Response Analysis of Soil-Structure Systems. EERC. University of California.

Seed H. (1970). Soil Moduli and Damping Factors for Dynamic Response Analysis. Report EERC70-10. University of California. Berkeley.

Seed H. (1979). Considerations in the Earthquake Resistant Design of Earth and Rockfill Dams. Geothecnique 29. Pag. 213-263.

USBR. (1976). Design of Gravity Dams. Denver, Colorado.

Villarraga M, (1987).Respuesta Aleatoria Tridimensional de Presas de Tierra. Tesis DEPFI-UNAM. México.

Villegas F. et al (1987), Evaluation of Dynamic Soil Parameters in Residual Soils Using Cyclic Test. VII CPMSIF-PCSMFE. Cartagena Colombia.

Vucetic M., Dobry R. (1991). Effect of Soil Plasticity on Cyclic Response. JGE. ASCE. 112,No. 11.

Walpole, R.E., Raymond, A.M. (1968). Probability and Statistics for Engineers and Scientists. The McMillan Company. New York.