Geotechnical and Geophysical Site Characterization 4 – Coutinho & Mayne (eds)
© 2013 Taylor & Francis Group, London, ISBN 978-0-415-62136-6

Centrifuge tests to evaluate the Po river bank seismic response

D. Giretti & V. Fioravante
University of Ferrara, Ferrara, Italy

L. Martelli
Emilia Romagna Region, Geological, Seismic and Soil Survey, Italy

ABSTRACT: On the behalf of the Italian Department for the Civil Protection, the Po River Basin Authority carried out a research project to evaluate the seismic hazard of about 90 km of the right bank of the Po river, from Boretto (Reggio Emilia Province) to Ro (Ferrara Province). The project provides the evaluation of 1) the regional seismic risk of the areas under study, 2) the bank and foundation soil properties, 3) the local seismic response of the ground foundation, 4) the liquefaction potential and 5) the evaluation of the static and dynamic bank stability. This paper presents the preliminary results of centrifuge tests carried out on model embankments to study their seismic response and dynamic stability.

Keywords: seismic centrifuge, bank, seismic amplification

1 INTRODUCTION

The banks of the Po river have strategic importance for the purpose of civil protection. Thus the Po River Basin Authority (AdBPo) was financed by the Italian Department for the Civil Protection to carry out a research project to evaluate the seismic hazard of about 90 km of the right bank of the Po river, from Boretto (Reggio Emilia Province) to Ro (Ferrara Province). The AdBPo involved in the project the Po River Interregional Agency (AIPO); the Seismic Surveys of the Lombardia and Emilia-Romagna Regions (RL & RER); the National Research Council (CNR); the National Geophysics and Vulcanology Institute (INGV); the Universities of Bologna (UniBO), Ferrara (UniFE), Firenze (UniFI), Milan (PoliMI) and Siena (UniSI); the Geotechnical Modeling Experimental Institute (ISMGEO).

The assessment of the seismic hazard has been conducted through several steps:

- analysis of the regional seismic hazard via statistical disaggregation, considering standard and macro-seismic approaches; definition of the reference response spectra and identification of compatible recorded accelerograms (CNR, UniSI);
- site characterization of the 90 km of right bank of the Po river under study via a large series of in situ tests: 300 CPTUs 35 m deep; 72 boreholes from 10 m to 150 m deep with undisturbed samples for advanced laboratory tests; 28 down hole and cross hole tests from 30 m to 150 m deep; 3 MASWs, 3 ReMi and 3 seismic refraction wave tests; monitoring of the environmental vibrations; installation of piezometers in boreholes. The in situ test program was designed and supervised by the RER;
- standard and advanced laboratory tests on undisturbed samples (AIPO);
- local seismic response analyses (UniFE, PoliMI);
- evaluation of the liquefaction potential (PoliMI);
- evaluation of the static and dynamic bank stability (UniBO & UniFI).
- evaluation of seismic response and seismic stability of the banks via dynamic centrifuge tests (ISMGEO, UniFE).

This paper presents the preliminary results of the centrifuge seismic tests carried out on model embankments to study their seismic response and dynamic stability.

The physical model tests were carried out using a single degree of freedom shaking table installed in the Italian centrifuge at ISMGEO. The bank models were reconstructed using silty and sandy soils retrieved in situ during the boreholes realized at one of the investigated sites (Casaglia, Ferrara Province), which was assumed as reference site. Two soil profiles and two geometries were tested. For each model four tests were carried out, using four different input motions, applied at the bottom of the model levee.

As input motions, the results of a seismic ground response analysis carried out referring to the Casaglia site were employed.

The seismic ground response analysis (under free field conditions) was performed using a mono-dimensional numerical code (ProShake) which accounts for the non-linear soil behavior through the equivalent linear method. As input motions at the bedrock, the accelerograms defined by the regional seismic hazard analysis for the Ferrara region were used. The soil stratigraphy and properties of the Casaglia site were define thanks to two 150 m deep boreholes and to a 150 m deep cross-hole test.

2 GEOLOGIC SETTING

The 90 km of the Po river banks from Boretto to Ro included in the research project are evidenced in Figure 1.

The main tectonic structure of the whole area is a buried ridge known as the Ferrara Folds, which reaches its maximum height NW the city of Ferrara under the Po river, near the site of Casaglia.

The subsoil is characterized by alluvial deposits of different depositional environments, which consist in an alternating sequence of silty-clayey layers of alluvial plain and sandy horizons of channel and levee, sometimes with fine conglomerates (Martelli et al. 2011).

These deposits are more than 500 m thick near Boretto, where the geological substratum consists of marine and transition deposits of lower-middle Pleistocene age. The thickness of the deposit reduces progressively towards Ferrara, where they reach their minimum depth (110–120 m) due to the buried structural high of the Ferrara folds, then it increases towards the delta mouth.

Geophysical test results and the values of fundamental frequencies of deposits obtained from ambient vibrations measurements, indicate that the depth of the seismic bedrock is about 80 m in the western area (Reggio Emilia) and about 110–120 m in the eastern region (Ferrara).

The regional seismic hazard study indicates relatively low levels of ground shaking. The seismic risk increases from west to east: in the eastern sector the greater contribution to seismic hazard is given by relatively small local earthquakes (magnitude around 5 within few tens of km) while in the western sector strong distant earthquakes (magnitude > 6 and distances of the order of hundred kilometers or more) are expected (Albarello 2009; Marcellini et al. 2010).

3 SITE INVESTIGATION

The 90 km of the Po river banks under study have been deeply investigated through a large series of in situ tests in order to define the geotechnical properties of the levees and the subsoil.

They consisted of 300 CPTUs 35 m deep, 66 boreholes from 10 m to 50 m deep with undisturbed samples for advanced laboratory tests; 3 couples of 150 m deep boreholes with undisturbed samples for advanced laboratory tests; 25 down hole tests from 30 m to 50 m deep; 3 cross-hole tests 150 m deep; 3 MASWs, 3 ReMi and 3 seismic refraction wave tests; monitoring of the environmental vibrations; installation of piezometers in boreholes (Martelli et al. 2011).

The tests were arranged in groups of 3 or 5 and they were realized along sections nearly perpendicular to the bank. The distance between two consecutive section was 1 km. Along each section at least on test was realized on the top of the bank, one at the ground level, one in the floodplain. As an example, Figure 2 shows a plane view of a test section, near the city of Ferrara, where 3 CPTUs, 35 m deep, were realized. Figures 3 and 4 report the measured profiles of the cone resistance, q_t and excess pore pressure, u_2, as a function of the depth z. The cross section of the levee and the stratigraphy up to the maximum depth reached (evaluated via the empirical correlation by Robertson 1990) are also reported. The Figures evidence the presence of two main stratigraphic units within the depth

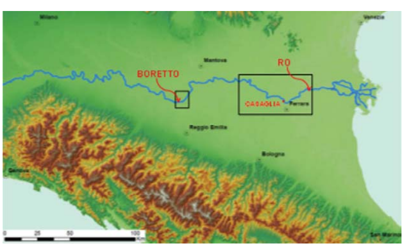

Figure 1. 90 km of the Po river banks, from Boretto to Ro, included in the research project.

Figure 2. Plane view of the levee near the reference site of Casaglia and position of 3 CPTUs.

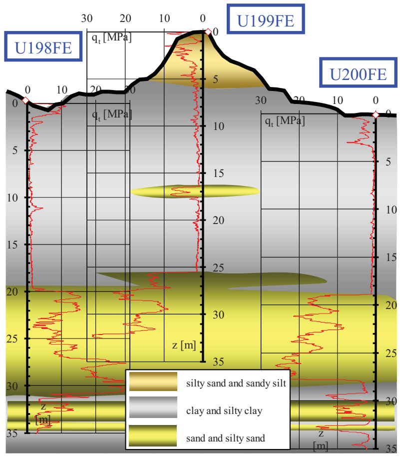

Figure 3. Cross section of the bank, profiles of the measured cone resistance q_t and soil stratigraphy.

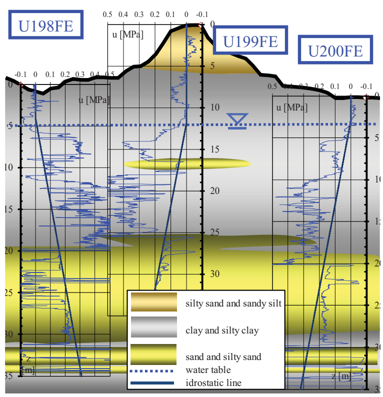

Figure 4. Cross section of the bank, profiles of the measured excess pore pressure u_2, soil stratigraphy and water table.

of 30–35 m, a shallower fine grained layer and a deeper coarse grained horizon. These units have been found along the whole river track under study, but they are characterized by different depth and thickness. The soil of the bank mainly consists of overconsolidated sandy silt and silty sand.

4 GROUND RESPONSE ANALYSIS OF THE FOUNDATION SOIL

The reference site of the seismic centrifuge tests on model levees presented in this paper is Casaglia, located north-west of the city of Ferrara (see Fig. 1). The input motions of the shaking tests were derived from a free-field seismic ground response analysis carried out for this site.

The seismic ground response was studied using the code Proshake (evolved version of Shake91, Idriss & Sun 1992) that performs 1D dynamic analysis in the frequency domain. The non-linear soil behavior and the strain dependent damping are accounted for through the equivalent linear method, that gives a reasonable estimate of soil response for moderate levels of shearing intensity and provided that no significant pore water pressure develops during seismic shaking. Six real ground acceleration records selected for the Ferrara region via the seismic hazard analysis (CNR & UniSI) were used as input motions of the analyses (Albarello 2009, Marcellini et al. 2010). The input accelerograms were assumed to be applied to a rock outcropping and deconvoluted to the seismic bedrock. The main characteristics of the input motions are summarized in Table 1, where PGA is the peak ground acceleration, PGV is the peak ground velocity, PGD is the peak ground displacement, d_{90} is the significant duration (Trifunac & Brady 1975), I_A is the Arias Intensity (Arias 1969) and s_i is the Housner Spectrum Intensity evaluated on the pseudo-acceleration spectrum PSA (Housner 1952).

Table 2 summarizes the stratigraphy of the Casaglia site, as resulted from a couple of 150 m

Table 1. Characteristic of the real earthquake records used as outcropping motions in the ground response analysis.

N. [–]	Code [–]	PGA [g]	PGV [m/s]	PGD [m]	d_{90} [s]	I_a [m/s]	s_i [m/s]
1	000200xA	0.22	0.14	0.01	10.89	0.73	5.30
2	000649yA	0.38	0.11	0.01	3.19	0.34	2.81
3	000651xA	0.22	0.10	0.01	5.42	0.21	2.71
4	000651yA	0.18	0.09	0.01	5.05	0.27	2.74
5	000829xA	0.34	0.14	0.01	1.27	0.20	3.08
6	006277xA	0.37	0.18	0.02	4.31	1.32	5.15

Table 2. Soil profile, shear wave velocity and natural unit weight at the Casaglia site.

Depth [from (m)–to (m)]	Soil type [–]	V_S [m/s]	γ_n [kN/m^3]
0–15	Clay—silty clay	170	18
15–31	Sand—silty sand	220	18
31–39	Sand—silty sand	270	18
39–65	Clay—silty clay	300	18
65–80	Sand—silty sand	400	18
80–101	Clay—silty clay	500	19
101–110	Clay—silty clay	500	19
110–130	Clay—silty clay—silty sand	900	22
>130	Miocene marl	≥1200	22

deep soundings, and reports the profile of the shear wave velocity V_S, measured by a 150 m deep cross-hole test realized in the boreholes. The Table shows that the substratum of the alluvial succession, made by sandy marine sediments Lower Pleistocene in age, was drilled at the depth of 110 m and Miocene marl, top of the pre-Quaternary succession of the Ferrara Folds, was drilled at the depth of 130 m. The shear wave velocity increases as the depth increases and reaches higher values than 800 m/s in the Quaternary marine substratum and 1200 m/s at the marl roof. In the ground response analysis the seismic bedrock was placed at the depth of 120 m, intermediate value between the depth of the sandy marine sediments and the marl roof. The shear modulus reduction curves and the damping curves used in the analyses were measured in resonant column tests on undisturbed samples and are reported in Figure 5, as functions of the strain level.

The results of the ground response analysis for the input motion N.6 are reported in Figures 6 and 7, where the computed time history of acceleration and the Fourier amplitude spectrum are represented, respectively. The main characteristics of the computed earthquake (named 6*) are given in Table 3. The predominant frequency is 3.2 Hz.

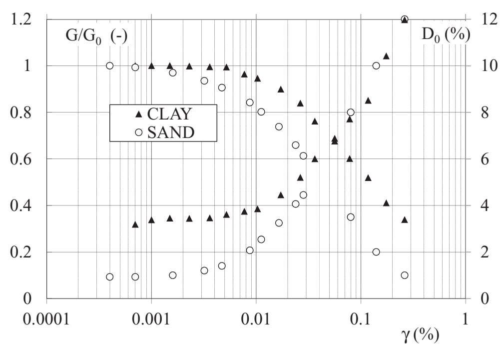

Figure 5. Shear modulus reduction curves and damping increment curves.

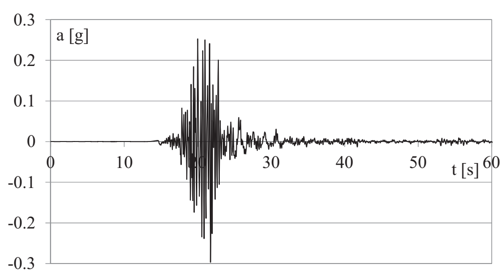

Figure 6. Ground response analysis: time history of accelerations computed at the soil surface (input motion N.6).

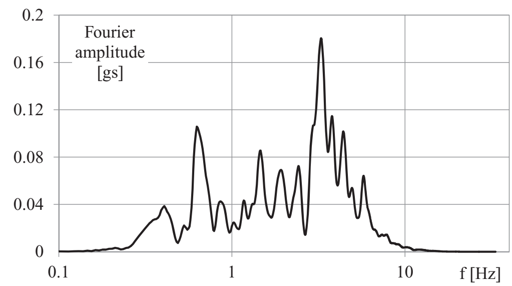

Figure 7. Ground response analysis: Fourier amplitude spectrum of the motion computed at the soil surface (input motion N.6).

Table 3. Ground response analysis: characteristic of the computed motion at the soil surface (input motion N.6).

N. [–]	code [–]	PGA [g]	PGV [m/s]	PGD [m]	d_{90} [s]	I_a [m/s]	s_i [m]
6*	006277xA -ampl	0.30	0.23	0.04	4.17	1.02	6.14

5 CENTRIFUGE TESTS

5.1 *The IMGEO seismic geotechnical centrifuge*

The model tests were performed using the ISMGEO Geotechnical Centrifuge (IGC), which has a symmetrical 2 m high, 1 m wide rotating arm with a radius of 3 m and a nominal radius of 2.2 m. The arm has two swinging platforms that hold the model container and the counterweight; during the test, the platforms lock horizontally to the arm to prevent the working loads from being transmitted to the basket suspensions. An outer fairing covers the arm and they concurrently rotate to reduce air resistance and model perturbation during flight. The centrifuge can reach a maximum acceleration of 600 g with a payload of 400 kg (further details can be found in Baldi et al. 1988). It has been recently equipped with a 1 degree of freedom shaking table, installed on the rigid rotating arm. The table works under an acceleration field up to 100 g, it can provide excitations at frequencies up to 1000 Hz and acceleration up to 50 g and it is able to reproduce real earthquakes at the model scale.

5.2 *Model banks, instrumentation, testing procedure*

The adopted geometrical scaling factor of the models is N = 50. All the bank models were tested under an acceleration field of a = 50 g, which was reached in correspondence of the bottom of the levees. The similarity relationships between model and prototype relevant for the presented tests are listed in Table 4 (Schofield 1980).

The models were reconstructed within a rigid container under plain strain conditions; the cross section of each model had a symmetric trapezoidal shape, as sketched in Figure 8. Since shaking in the x direction only, the container was designed to confine the embankments only in the y direction and the two slopes did not touch the connection end walls. To minimize the side friction, the lateral walls were treated with a layer of oil. A rough layer was applied on the container bottom.

Four different models were tested: two geometries and two soil profiles of the levees were adopted, as evidenced in Figure 9. The model dimensions are listed in Table 5. The models were prepared in horizontal layers 2 cm thick by tamping, then they were cut to form the slopes at the α° values to the horizontal container bottom. The sand and the silt used to reconstitute the models were retrieved in situ during soundings. The soil properties are listed in Table 6; they were measured via laboratory tests on samples taken from the centrifuge models at the end of the tests. The values of the natural unit weight, γ_n water content, w and void index, e in the table refer to the mean test conditions.

Table 4. Similarity relationships.

Quantity	Prototype	Model
Length, L	N	1
Velocity (projectile), v	1	1
Acceleration, a	1	N
Mass, m	N^3	1
Force, F	N^2	1
Stress, σ	1	1
Strain, ε	1	1
Mass density, ρ	1	1
Time (dynamic), t_d	N	1
Frequency, f	1	N

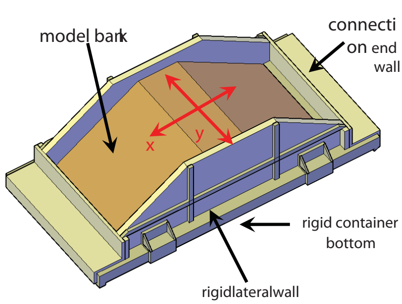

Figure 8. Scheme of the model bank and of the rigid container.

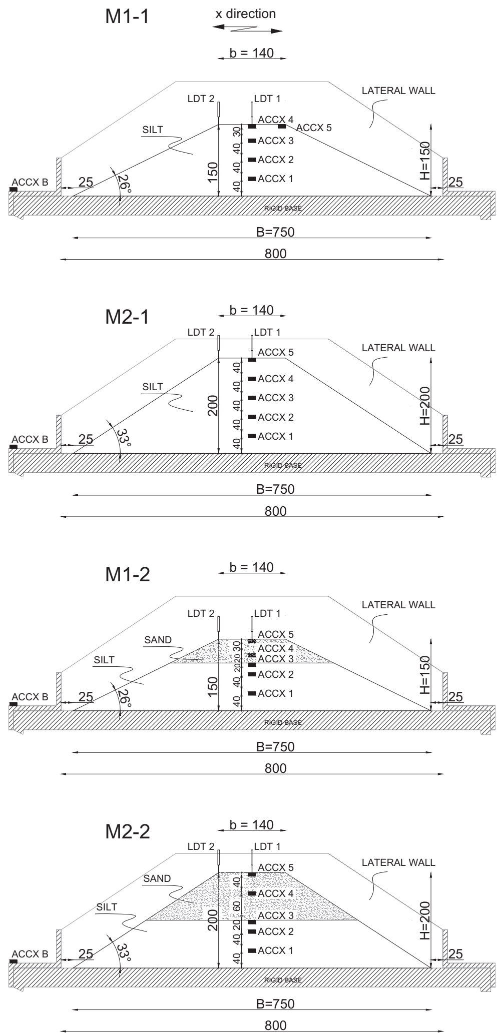

Figure 9. Model configurations and dimensions. All dimensions in mm.

The models were instrumented with six miniaturized accelerometers (Fig. 9), one installed at the container bottom (ACCX B) and five embedded within the model banks, at variable heights from the bottom (ACCX 1–5). The accelerometers were arrange to measure acceleration in the x direction. Two displacement transducers LDT (hold by a rigid frame connected to the lateral walls) were placed on the top of the embankments to measure the crest vertical settlements. During the tests the instrumentation measures were recorded at a frequency of 5000 Hz. Two pictures of a model embankment and details of the instrumentation are show in Figure 10.

After model preparation, the container was placed into the centrifuge, whose speed was slowly

Table 5. Model dimensions. Prototype and model scale.

		B		b		H		α
Model	Soil	Mod [mm]	Prot [m]	Mod [mm]	Prot [m]	Mod [mm]	Prot [m]	[°]
M1-1	Silt	750	37.5	140	7	150	7.5	26
M1-2	Silt/sand							
M2-1	Silt	750	37.5	140	7	200	10	33
M2-2	Silt/sand							

B = length of the bank base; b = length of the bank crest; H = bank height.

Table 6. Test soil properties.

	D_{50} [mm]	Uc [–]	γ_n [kN/m^3]	W [%]	E [–]	ϕ'_{cv} [°]
Silt	0.025	35.2	15.4	10	0.9	30
Sand	0.39	3.16	16	5	0.7	35.4

D_{50} = mean grain size; U_C = uniformity coefficient; γ_n = natural unit weigh; w = water content; e = void index; ϕ'_{cv} = shearing resistance angle at critical state.

Figure 10. Pictures of model M2-2 and instrumentation.

increased to reach the acceleration target value of 50 g. After the model consolidated under its weight, the input accelerogram was triggered.

Sixteen dynamic tests were performed: each model was tested under four of the six accelerograms computed at the soil surface via the ground response analysis; the time histories of acceleration were properly scaled, i.e. $a_{mod} = 50a_{prot}$, $t_{d,mod} = t_{d,prot}/50$, $f_{mod} = 50f_{prot}$. During each test, before triggering the input accelerogram, the noise and the ambient vibrations measured by the accelerometers were recorded for few seconds.

5.3 *Model 1-1—input motion 6**

In the following section the results of the test on the model M1-1, subjected to the earthquake 6* are reported; all the results are plotted at the prototype scale. Figure 11 shows the Fourier amplitude spectrum of the noise measured at the top of the embankment by ACCX 4, normalized to the Fourier amplitude spectrum of the noise measured at the base of the embankment by ACCB. The measures were recorded before triggering the input accelerograms. The spectral ratio shows that the bank amplifies all the frequency in the range 3.5–25 Hz, with a peak at the frequency f = 9 Hz, which is likely to be the natural frequency of the model.

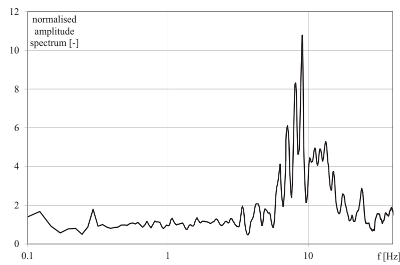

Figure 11. Model M1-1. Normalized Fourier amplitude spectrum of the noise pre-event.

Figure 12 shows the time histories of acceleration along the height of the bank and evidences that the acceleration increases as the distance from the base increases, due to amplification effects of the embankment.

The amplification effects of the embankment are also evidenced in Figure 13 where the Fourier amplitude spectra of acceleration are reported. The amplification is significant in the range of frequency of 2.5–25 Hz and increases from the base towards the crest. The measures of top accelerometers (ACCX 4 and ACCX 5) evidence slightly higher amplification effects at the crest center than at the edge.

Two amplification peaks at the frequencies 5.4 Hz and 6.3 Hz are clearly evidenced by the amplification functions (Fourier amplitude spectra of the accelerations measured within the models normalized to the Fourier amplitude spectra of the input acceleration measured at the base)

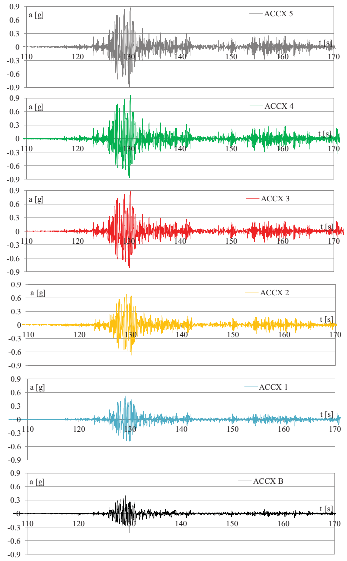

Figure 12. Model M1-1, input motion N.6*. Time histories of the acceleration along the bank height.

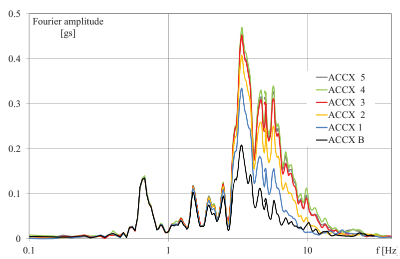

Figure 13. Model M1-1, input motion N.6*. Fourier amplitude spectra.

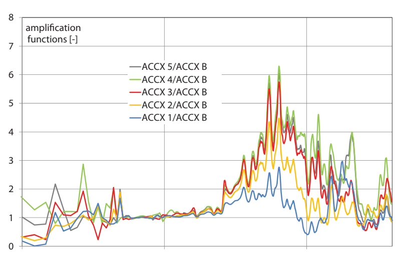

Figure 14. Model M1-1, input motion N.6*. Amplification functions.

represented in Figure 14. The reduction of the natural frequency from 9 Hz to 5–6 Hz could be attributed to stiffness decay and damping increment as the deformations increase during the earthquake.

As recorded by the displacement transducers LDT1 and LDT2 (see Fig. 9), very small crest settlement were measured during shaking, about 0.22 mm at the prototype scale and no sign of local instability phenomena were observed in the embankment during and after the test.

The amplification phenomena observed were quantified via the amplification factors defined below:

$$FA_{0.1-0.5} = \frac{s_{i,0.1-0.5}(PSA_{ACCX4})}{s_{i,0.1-0.5}(PSA_{ACCXB})} \tag{1}$$

$$FA_{0.5-1.5} = \frac{s_{i,0.5-1.5}(PSA_{ACCX4})}{s_{i,0.5-1.5}(PSA_{ACCXB})} \tag{2}$$

where: $s_{i,0.1-0.5}(PSA)$ = is the Housner intensity of the pseudo-acceleration spectrum between the periods 0.1 s–0.5 s; $s_{i,0.5-1.5}(PSA)$ = is the Housner intensity of pseudo-acceleration spectrum between the periods 0.5 s–1.5 s. The computed values of the amplification factors are listed in Table 7, where the peak ground accelerations measured at the model base (ACCX B) and top (ACCX 4) are also reported. The pseudo-acceleration spectra are shown in Figure 15. The amplification is higher between the periods 0.1 s–0.5 s and the FA value is greater than 2.

Figures 16 and 17 show for both the bank geometries M1 and M2 the profiles of the ampli-

Table 7. Model M1-1, input motion N.6*. Characteristic of the accelerograms measured at the model embankment base and crest and amplification factors.

N. [–]	code [–]	$PGA_{ACCX\ B}$ [g]	$PGA_{ACCX\ 4}$ [g]	$FA_{0.1-0.5}$ [m/s]	$FA_{0.5-1.5}$ [m/s]
6*	006277xA -ampl	0.425	0.958	2.66	1.29

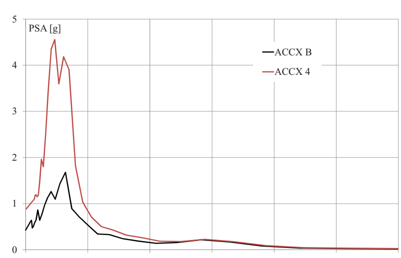

Figure 15. Model M1-1, input motion N.6*. Pseudo-acceleration spectra at the base and at the crest of the model embankment.

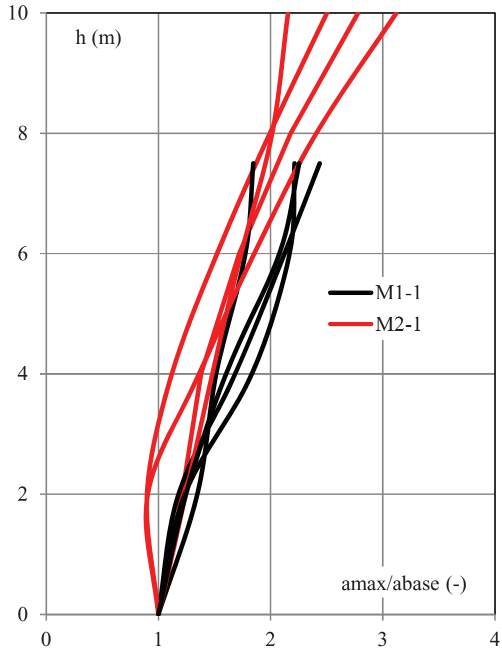

Figure 16. Amplification ratio profiles for all the homogeneous models (M1-1 and M2-1).

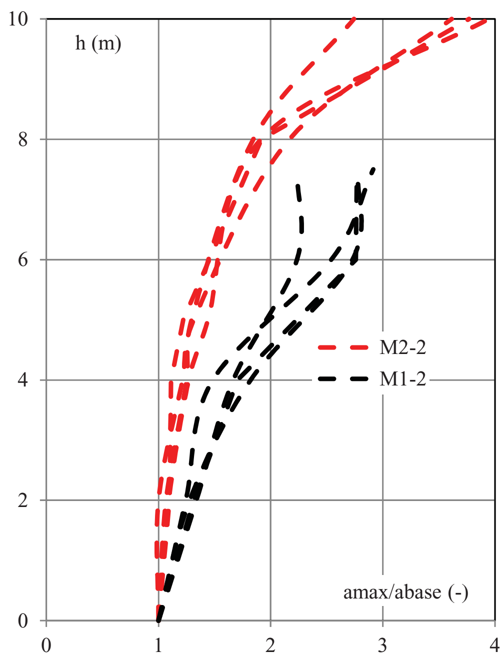

Figure 17. Amplification ratio profiles for all the stratified models (M1-2 and M2-2).

fication ratio a_{max}/a_{base}, where a_{max} and a_{base} are the maximum accelerations measured along the bank center line and at the base (the latter measured by the base accelerometer). The results for all the input motions and for both the homogeneous and stratified models are shown.

All the motions and models gave similar profiles, characterized by amplification effects along the whole bank height; the maximum values of the amplification ratios were obtained at the crest of the embankments. The amplification is influenced by the stratification, especially near the top.

6 FINAL REMARKS

The preliminary results of centrifuge seismic tests carried out on model embankments to study their seismic response and dynamic stability are presented. This study is part of a research project to evaluate the seismic hazard of about 90 km of the right bank of the Po river, which have included extensive site characterization.

The centrifuge shaking table tests were performed using as input motions the results of a seismic ground response analysis carried out referring to one of the site investigated via in situ and laboratory tests. The soils retrieved from the real banks were used to reconstruct the models. Four models (two geometries and two soil profiles) subjected to four time histories were tested (in total sixteen tests).

The tests showed a significant tendency of the banks to amplify the accelerations. The amplification is significant in a wide range of frequencies (from about 2.5 to 25 Hz) and increases from the base towards the crest. The amplification is influenced by the stratification, especially near the top of the embankments, also due to wave focalization.

A reduction of the natural frequency of the embankments during the tests was observed, probably due to the stiffness decay and damping increment, as the deformations imposed by the input motions increase. No signs of local instability were observed.

Test interpretation is still ongoing.

Tests results will be used as benchmark for the calibration of numerical dynamic analyses.

ACKNOWLEDGMENTS

The research reported herein was financed by the Po River Basin Authority, whose support is gratefully acknowledged. The precious assistance in processing acceleration data of Dr. Marcellini and Dr. Tento (National Research Council) is also acknowledged. Thanks to the ISMGEO staff and to Mr. Sergio Airoldi for assistance in centrifuge works.

REFERENCES

Albarello, D. 2009. Stima della pericolosità sismica dell'argine meridionale del Po sulla base di una analisi statistica delle storie sismiche di sito. Report. Dipartimento di Scienze della Terra Università degli Studi di Siena.

Arias, A. 1969. A measure of earthquake intensity. Seismic design for nuclear power plants. R.J. Hansen ed., Massachusetts Institute of Technology, 489 pp.

Baldi, G., Belloni, G., Maggioni, W. 1988. The ISMES Geotechnical Centrifuge. In Centrifuge 88, Paris, Corté J.F. Ed., Balkema, Rotterdam, pp. 45–48.

Housner, G.W. 1952. Spectrucm Intensities of strong motion earthquakes. Proceeding of the symposiucm on Earthquake and Blast effects on Structures. Earthquake Engineering Research Institute, 322 pp.

Idriss, I.M. & Sun, J.I. 1992. "SHAKE91: A computer program for conducting equivalent linear seismic response analyses of horizontally layered soil deposits," User's Guide, University of California, Davis, California, 13 pp.

Marcellini, A., Albarello, D., Gerosa, D. 2010. Accelerogrammi di riferimento per l'argine destro del Po. Report. Istituto per la Dinamica dei Processi Ambientali, Dipartimento di Scienze della Terra Università degli Studi di Siena.

Martelli, L., Severi, P., Biavati, G. and Rosselli, S. 2011. Modello geologico per le verifiche di stabilità in condizioni sismiche dell'argine destro del Po tra Boretto (RE) e Ro (FE). Report. Soil, Seismic and Geological Survey.

Robertson, P.K. 1990. Soil Classification using the cone penetration test. *Canadian Geotechnical Journal* 27(1): 151–158.

Schofield, A.N. 1980. Cambridge Geotechnical Centrifuge Operations. Geotechnique, Vol. 24, No. 4, pp. 227–268.

Trifunac M.D. & Brady A.G. 1975. A study on the duration of strong ground motion. Bull. Seism Soc. Am., 65.