HYDRAULIC ANALYSES FOR A NEW BRIDGE OVER THE PARANA RIVER, ARGENTINA

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ABSTRACT

This paper summarizes hydraulic studies carried out as part of the feasibility study for a new bridge over the Parana River in Argentina. The bridge will physically connect the provinces of Santa Fe and Corrientes, currently separated over the length a 200-km river reach. In addition to the hydraulic studies summarized here, the feasibility study includes analyses of the geology and geomorphology of the sites, land surveying, geophysical and hydrologic studies, studies of the navigation channel and fluvial transportation, transportation studies, and environmental impact studies. The main components of the present study were a geomorphologic analysis of the middle reach of the Parana River, and a quantitative hydraulic study. Qualitative and quantitative analyses provided a description of the hydraulic system, and served as a basis for a comparison of alternative bridge locations and the analysis of their vulnerability. The quantitative hydraulic study included the construction, calibration, and operation of one-dimensional (1D) and depth-averaged two-dimensional (2D) flow models. In order to tie the models to the most reliable data the 1D models covered a 450-km long reach, and included the main channel (about 2 km wide) and the flood plain (about 30 km wide). The results of the 1D models played a major role in the selection of the bridge location, and were subsequently used as boundary conditions for 2D modeling. The 2D models were constructed for a 70 km x 30 km modeling domain. This paper presents results of 1D modeling for large floods, both in the natural state and with the proposed bridge, and results of 2D modeling in the natural state. Modeling at this scale requires sound judgment to strike a balance between the desirable level of model detail and the cost of acquiring data for model construction and calibration.

Key Words: Hydraulic analyses, Bridge, Parana River, Feasibility study

1 INTRODUCTION

The Parana River in South America ranks as the sixth largest in the world in terms of average discharge (22×10^3 m^3/s according to Schumm and Winkley, 1994), and its Argentinean reach stretches from Puerto Iguazu on the north to the Rio de la Plata estuary in the south (Fig. 1). Its estimated average suspended load, bed load, and total load are 23×10^6, 2.2×10^6, and 25.2×10^6 metric tons per year, respectively (Paoli and Schreider, 2001.) The economic development of northeastern Argentina is constrained by the long distances between bridges over the Parana. More specifically, the provinces of Santa Fe and Corrientes share a 200-km river reach (defined by the sections labeled as Bella Vista and Esquina in Fig. 1), yet there is no bridge between those provinces.

The Argentinean Federal Investment Council is funding a feasibility study for a bridge between Santa Fe and Corrientes. The feasibility study includes analyses of the geology and geomorphology of the sites, land surveying, geophysical studies, hydrologic and hydraulic studies, studies of the navigation channel and fluvial transportation, transportation studies, and environmental impact studies.

The selection of a bridge location in a large river is a non-trivial task. Seven alternative (and 29 sub-alternative) bridge locations were studied in the prescribed 200 km reach. The selection had to consider several, sometimes conflicting criteria. General optimization criteria included minimizing total

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transportation costs, integration with railway and fluvial port systems, maximizing the financial feasibility of construction, and improving regional development and quality of life. Specific hydraulic criteria included the geomorphologic stability of the main channel, islands, and alluvial valley; the hydraulic design of the main and auxiliary bridges; and the minimization of additional flooding, of perturbation to the navigation channel, and of both general and local erosion.

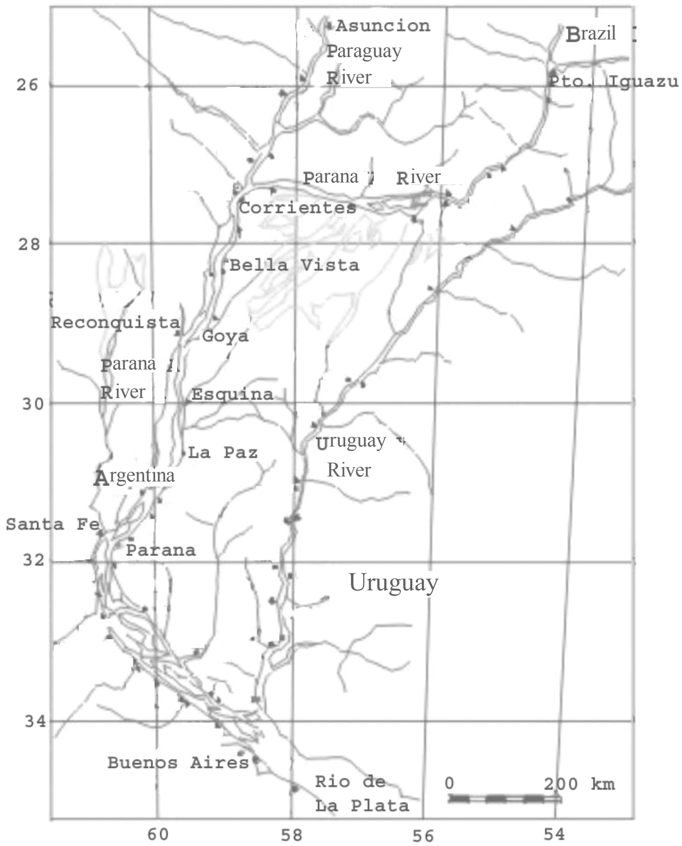


Fig. 1 Location of the study

The middle reach of the Parana River has been the subject of numerous hydraulic studies (e.g., Bombardelli et al., 1997; Paoli and Schreider, 2000). The two main components of the present multidisciplinary hydraulic analysis were a geomorphologic study of the middle reach of the Parana River, and the hydraulic quantitative study. Qualitative and quantitative analyses provided a complete description of the hydraulic system, and served as a basis for a comparison of alternative bridge locations and the analysis of their vulnerability. This paper briefly describes the geomorphologic study and focuses on the quantitative hydraulic study.

The quantitative hydraulic study started with the construction and calibration of one-dimensional (1D) models using the MIKE11 (Havno et al., 1995) and HEC-RAS 3.0 (USACE, 2001) packages. In order to tie the models to the most reliable data the 1D models were applied to a 450-km long reach between the sections labeled Corrientes and La Paz in Fig. 1. The 1D models were supported by some 30 measured cross sections that include the main channel (about 2 km wide) and the flood plain (about 30 km wide). Results of the 1D models were a key factor in selecting the North alternative of the Goya-Reconquista site as the bridge location, and were subsequently used as boundary conditions for two-dimensional (2D) modeling.

The 2D models were constructed using the RMA2 (USACE, 1997) and CYTHERE-ES1 (Benque et al., 1982) packages for a 70 km x 30 km modeling domain centered on Goya. This paper presents representative results of 1D modeling for large floods, both in the natural state and with the proposed bridge, and results of 2D modeling for the natural state.

2 ONE-DIMENSIONAL MODELS

Both the MIKE 11 (Havno et al., 1995) and the HEC-RAS (USACE, 2001) packages were used to validate results and to take full advantage of each software capability. The 1D approach is appropriate to model the whole river reach and to provide a basis for the comparison of alternative bridge locations. Both packages are based on the one-dimensional de Saint-Venant (1871) equations for unsteady flow. The equations are based upon the following assumptions:

(i) The flow is one-dimensional i.e. the velocity is uniform over the cross section and the water level across the section is horizontal.

(ii) The streamline curvature is small and vertical accelerations are negligible, hence the pressure is hydrostatic.

(iii) The effects of boundary friction and turbulence can be accounted for through resistance laws analogous to those for steady state flow.

(iv) The average channel slope is small so that the cosine of the angle it makes with the horizontal may be replaced by unity.

The two equations representing conservation of mass and momentum are respectively (Havno et al., 1995):

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \tag{1}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial\left(\beta \dfrac{Q^2}{A}\right)}{\partial x} + g\,A\frac{\partial h}{\partial x} + gA\frac{Q|Q|}{K^2} = 0 \tag{2}$$

where Q = discharge (m^3s^{-1}), A = flow area (m^2), q = lateral inflow (m^2s^{-1}), h = stage above datum (m), β = momentum distribution coefficient, K(h)= Q/ S_f =conveyance factor of the channel (m^3s^{-1}), and S_f = momentum slope. The conveyance factor K was estimated using Manning's empirical resistance law, with the roughness coefficient varying across the watercourse width. Following common practice, the compound channel shape was subdivided into main channel and floodplain subsections (vertical slices). The 2D modeling results presented below serve as a preliminary test of the validity of the de Saint Venant assumptions in this case.

Fig. 2 shows the comparison of water levels measured (shown by circles) and predicted (solid lines) at four cross sections depicted in Fig. 1, namely Bella Vista (top left), Goya (top right), Reconquista (bottom left), and Esquina (bottom right), for the 1983 long-duration flood. During that flood, the discharge exceeded 35,000 m^3/s during the first 7 months and exceeded 25,000 m^3/s during the rest of the year. The parameters determined by calibration were Manning's roughness coefficient, n, for the main channel and the floodplains. The estimated parameters were n = 0.025 for the main channel, and a value 4 times larger for the floodplains. The main factors affecting resistance to flow in the main channel are grain roughness, bed form roughness, and meandering, and the main factor affecting resistance to flow over the floodplains is vegetation. Forested areas with trees up to 40 m high exist on the floodplains. Flow in vegetated channels is an active area of research, but it may suffice to say that Chow (1959) reported values on n > 0.1 for channels that are not cleared for a number of years. The value of the roughness coefficient estimated for the main channel (approximately n = 0.025) is consistent with Amsler and Schreider's (1999) prediction of dunes height during floods of the Parana River.

The measured peaks were assigned the largest weight in the calibration because the bridge will be designed for large discharges. The good agreement observed at the hydrograph peaks, shown in Fig. 2, provided a reliable basis to compare the bridge effects at the alternative locations.

The capability of HEC-RAS to represent bridge and embankment details was used to better compare the flow in the river in the natural state and after bridge construction, and to estimate the distribution of discharge between the main channel and the floodplains. Fig. 3 shows representative results for the Goya-Reconquista site. For the 500-year and 1000-year floods (Q=75,000 m^3/s and 82,700 m^3/s, respectively) the estimated bridge backwaters are 0.12 m and 0.15 m, respectively, both smaller than the 0.30 m considered as acceptable by taking into account the elevation of flood-control structures in nearby Goya. The 1D models estimated an approximately 6 km - long backwater because of the bridge. The change in velocity distribution induced by the bridge was taken as representative of the vulnerability of each location and taken into consideration in the comparison of alternatives.

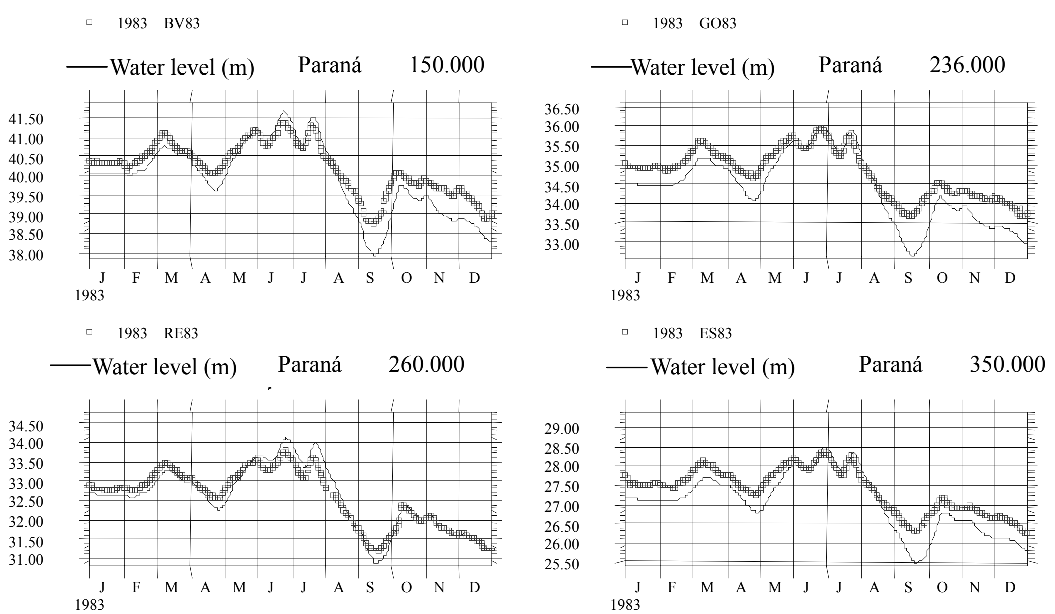
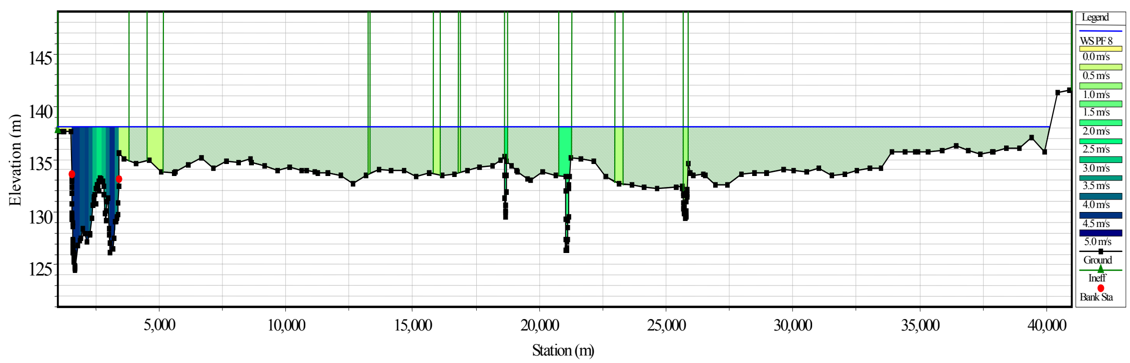
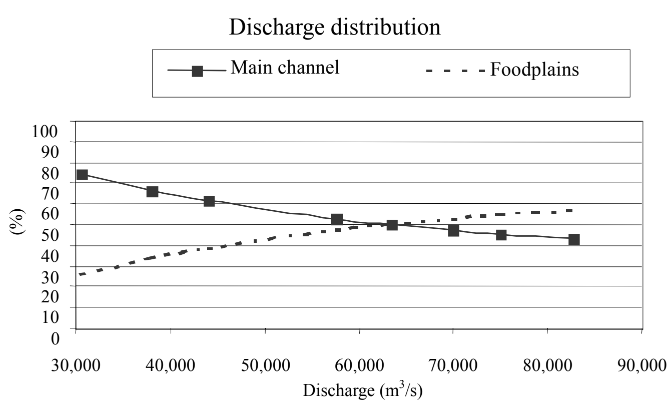


Fig. 2 Measured (circles) and calculated (solid lines) water surface levels for the 1982-83 flood at Bella Vista (top left), Goya (top right), Reconquista (bottom left), and Esquina (bottom right) (Q_p = 60,000 m^3/s.)



a



b

Fig. 3 HEC-RAS results for the selected location. a) Water surface level and velocity for the river after bridge construction. Colored areas denote flow areas under the main and auxiliary bridges, and hatched areas denote inactive areas closed by embankments. b) Discharge distribution.

3 GEOMORPHOLOGIC STUDY

The geomorphologic study included the comparison of 1970/76 navigation charts with 1994/98 (high resolution LANDSAT 5 and 7) satellite images, investigation of morphometric characteristics of the main channel and the alluvial valley, and analyses of the stability of the main channel, sediment transport, dominant discharge, and of the alternative bridge locations. The 1970/1998 time frame allowed analysis of the mobility of the Parana River over more than two decades, including the effects of big floods such as those of 1982-83, 1992, and 1998. Fig. 4 illustrates results of the geomorphologic study near the selected bridge location.

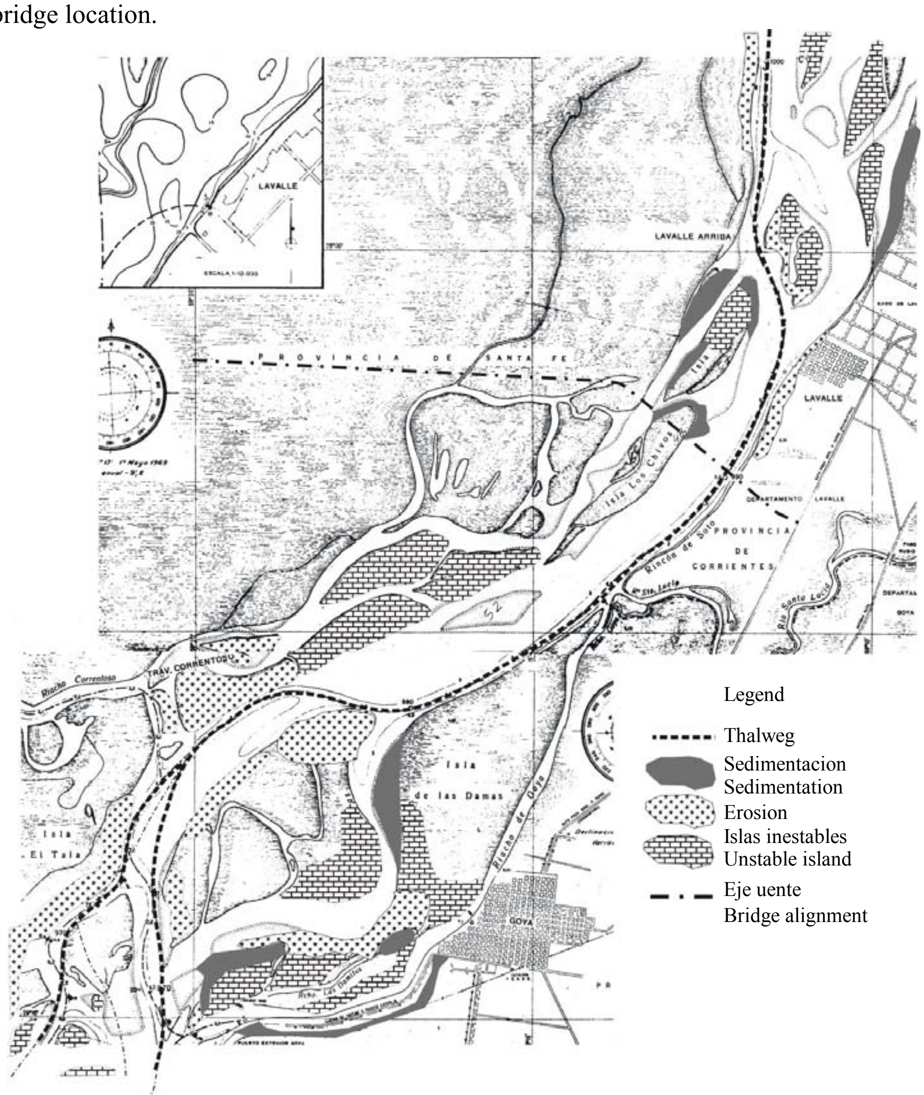


Fig. 4 Areas with observed deposition (green), erosion (brown), and unstable islands (blue) between 1970 and 1994. The line labeled "2.1.3" shows the selected bridge location

4 CONSTRUCTION OF TWO-DIMENSIONAL MODELS

The 2D models were constructed to obtain a more realistic prediction of the flow field near the selected bridge location. A 2D model solves the shallow water equations (Kuipers and Vreugdenhil, 1973). One can obtain the shallow water equations by averaging over the depth the Reynolds equations for time-averaged incompressible mean flow, and then modeling the resulting depth-averaged terms. The derivation procedure involves the hydrostatic pressure assumption, Leibnitz' rule for integrals, and the kinematic boundary conditions at the free surface and the bed. The depth-averaged continuity and momentum equations are

$$\frac{\partial(h+z_b)}{\partial \mathrm{t}}+\frac{\partial U_i}{\partial \mathrm{x}_\mathrm{i}}=0 \tag{3}$$

$$\frac{\partial U_i}{\partial \mathrm{t}}+\frac{\partial}{\partial \mathrm{x}_\mathrm{j}}(\widetilde{u}_j U_i)+gh\frac{\partial(h+z_b)}{\partial \mathrm{x}_\mathrm{i}}-\frac{\partial}{\partial \mathrm{x}_\mathrm{j}}\left[\widetilde{\nu}_t\,\frac{\partial(h\widetilde{u}_i)}{\partial \mathrm{x}_\mathrm{j}}\right]+\frac{1}{\rho}(\tau_{bi}-\tau_{si})=0 \tag{4}$$

where $h(x_i, t)$ = flow depth; $z_b(x_i, t)$ = bottom elevation; $\tilde{u}_i$ = depth averaged velocity; $U_i = h\tilde{u}_i$ = unit width discharge; $\widetilde{\nu}_t$ = depth averaged eddy (or turbulent) viscosity; ρ = fluid density; τ_{si} = surface shear stress; and τ_{bi} = bed shear stresses; all in the x_i direction. Equation (4) states, in depth averaged terms, the balance between changes in momentum flux per unit mass, gradients of the free surface elevation, surface and bed shear stresses, and divergence of depth averaged turbulent stresses. Depth averaged laminar stresses and dispersion terms are assumed negligible compared with depth averaged turbulent stresses, which are modeled in terms of the divergence of the gradients of the corresponding specific discharges (Benque et al., 1982). Bottom shear stresses are related to mean velocities via the usual friction laws (e.g., Chezy or Manning).

The more general shallow water equations have the potential to improve the flow description, but their application requires a corresponding improvement in the supporting data in terms of geometry, boundary conditions, and data for calibration. By decreasing the distance between surveyed sections one can improve model accuracy, but the surveying cost rises very rapidly for a large river. Considerable engineering judgment was necessary to decide the scope of additional surveying required, and to optimize the combination of existing information with technologies such as satellite imaging.

The 2D modeling domain is shown in Fig. 5a. Its width is some 30 km, approximately equal to the valley width. Its length is approximately 72 km, or 2.4 times the valley width, centered about the proposed bridge location. The selected length is intended to simulate the flow distribution between main and secondary branches and to minimize the influence of boundary conditions, and is much larger than the bridge backwater length estimated by the 1D models. Fig. 5a shows the topographic and bathymetric information generated for 2D modeling. The solid lines -and labeled sections- denote 5 sections surveyed in a previous study (a total of 30 sections was originally surveyed along the total river reach). The dark x's denote 5 full cross sections, 7 cross sections of the main channel near the bridge location, 7 cross sections across the Parana Mini secondary branch, and a longitudinal profile along the right bank of the main channel, all surveyed for this study. The gray x's denote interpolated sections, approximately 3 km apart from each other. The interpolated cross sections were superimposed on a satellite image from February 1998, when the discharge was Q = 33,000 m^3/s. The interpolated elevations were kept while the sections were displaced in the east or west direction until the main and secondary channels coincided with the satellite image. Ad-hoc interpolating software was applied to the geometric information shown in Fig. 5a to generate regular grids. Fig. 5b shows the grid generated for the RMA2 package; a simpler rectangular grid was generated for the CYTHERE-ES1 model. The SMS interface was used to generate two grids for the domain between the Pastoril and Los Vascos sections. The first, coarse grid consisted of 8,600 triangular elements (17,700 nodes). The typical element size was 300 m in the main channel and 1 km in the floodplains. A second, refined grid was constructed after an additional bathymetric survey (dark x's in Fig. 5a). The typical element size was 150 m in the main channel and the Parana Mini branch, and between 500 and 1000 m in the floodplains. The second grid had over 10,000 elements. Grid independence was verified by comparing modeling results obtained using both grids.

5 MODELING SCENARIOS, BOUNDARY CONDITIONS, AND ROUGHNESS

Predictions from the 1D model for the full river reach produced adequate boundary conditions for the 2D modeling domain. The 2D modeling scenarios consist in steady flow for the 1000-year, 500-year, and 50-year floods. The discharge at the upstream boundary was distributed between the main channel and the floodplain according to the 1D model predictions. The downstream boundary condition is a known, horizontal water surface level taken from the 1D model predictions. The left and right margins are impermeable boundaries. The edge of the floodplain is part of the 2D model solution, and that information is directly useful in model calibration. Simulations of the order of days were verified to be sufficiently long to reach steady state, and that the results are independent of the initial condition. Calculations were initially done using values of Manning's coefficient for the main channel and the floodplains obtained from the 1D model calibration. Fig. 6 shows a detailed roughness distribution estimated using satellite images and used to refine the 2D model.

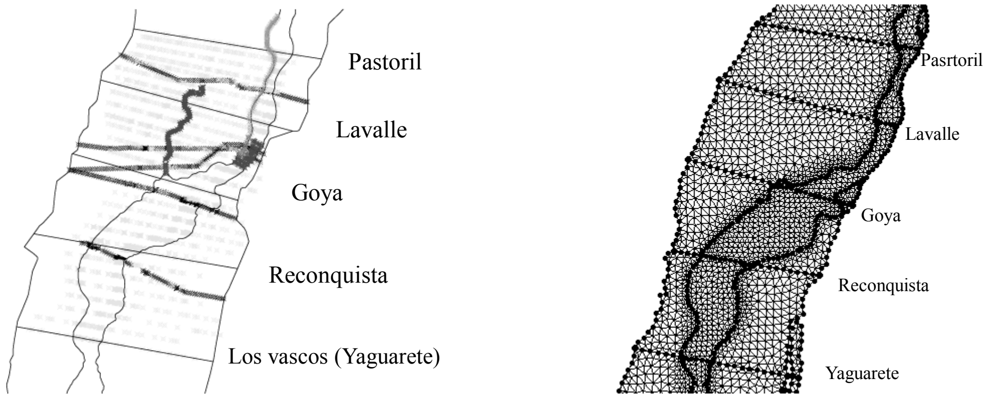


Fig. 5 a) Surveyed and interpolated cross sections in the 70 km x 30 km modeling domain centered on Goya; b) grid generated for the RMA2 model

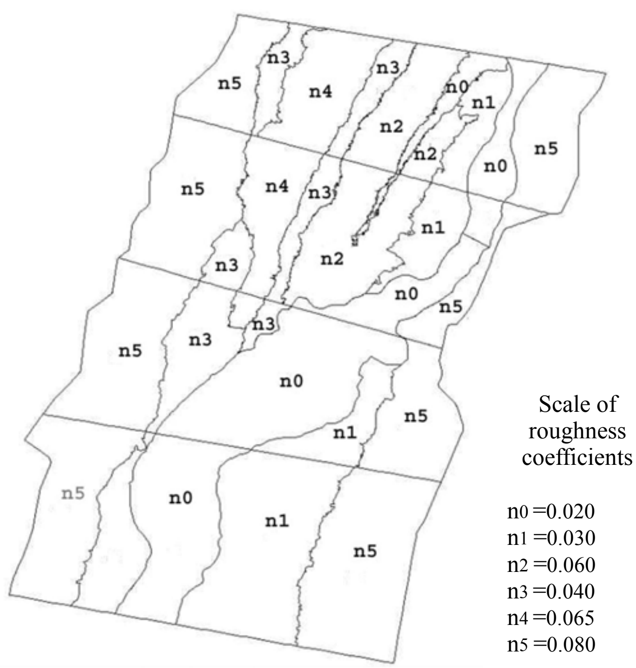


Fig. 6 Detailed roughness distribution estimated using satellite images

6 TWO-DIMENSIONAL MODELING RESULTS FOR THE RIVER IN ITS NATURAL STATE

Fig.s 7a and 7b show the water depth and water surface elevation, respectively, predicted by the RMA2 model for Q = 48,000 m^3/s (50 year flood). Fig. 8a is a satellite image obtained on May 5 1998, when the discharge was 48,000 m^3/s. Fig. 8b shows the predicted flow field (velocity magnitude and velocity vectors) for the same discharge. These figures show good agreement between observation and prediction in terms of the extent of flooded areas and the general flow pattern shown in false color in the satellite image. The figures also illustrate how the flow departs from the one-dimensional assumptions. The results obtained with the CYTHERE-ES1 model are very similar to the RMA2 results and are not presented because of space limitations.

7 ONGOING WORK

Ongoing work to model the effects of bridge construction is advancing at a slow pace because of the economic crisis in Argentina. A carefully selected set of velocity measurements will be taken near the bridge location. The 2D models are being upgraded by including the bridge and embankment geometries. The results of the upgraded 2D models will be used to estimate the bridge backwater, the effects of flooding on populated areas and vegetation, the influence of the flow pattern on navigation, the behavior of the channel-and-wetlands ecosystem, the erosion at bridge piers and abutments resulting from the predicted velocity field, the effect of the floodplain velocity field on sedimentation, and the possible formation of new islands.

8 CONCLUSIONS

The combination of a geomorphologic study and 1D modeling provided sufficient information to support sound decisions in the selection of the bridge location. 2D simulations of the river in its natural state showed good agreement with satellite image observations in terms of the extent of flooded areas and the general flow pattern. Modeling at this scale requires sound judgment to strike a balance between the desirable level of model detail and the cost of acquiring data for model construction and calibration.

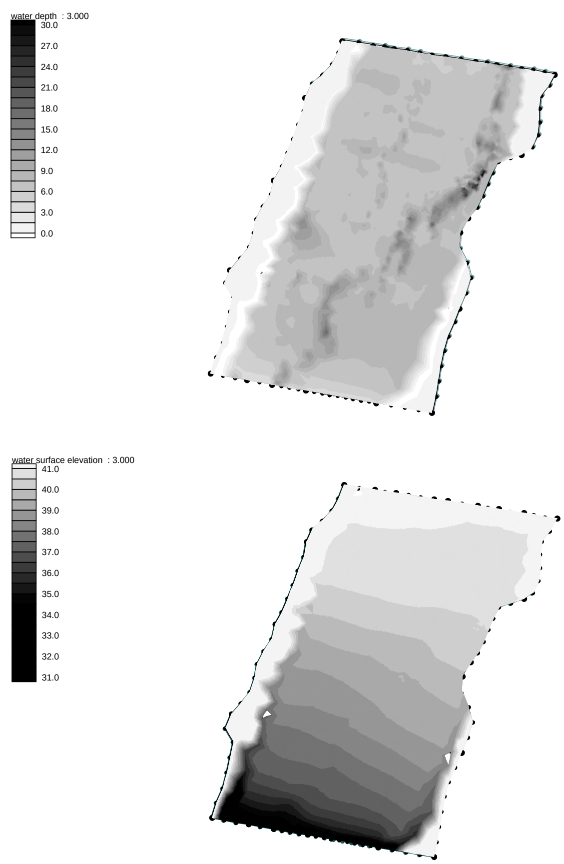


Fig. 7 a) Water depth (m); b) Water surface elevation (m) predicted for Q = 48,000 m^3/s

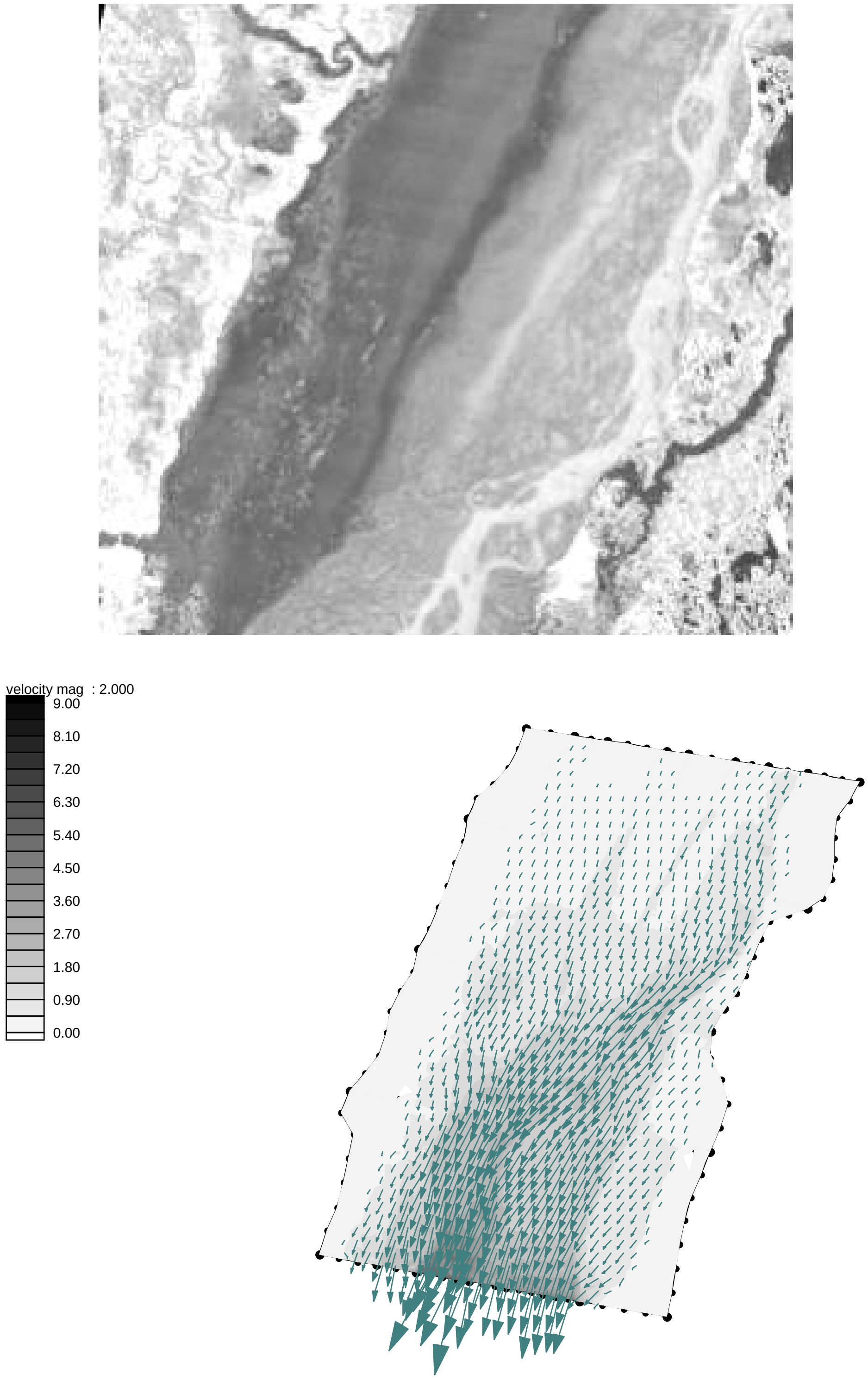


Fig. 8 LANDSAT 7 satellite image (5/5/98, Q = 48,000 m^3/s) showing flooded areas and general flow pattern in false color. b) Velocity field (magnitude and velocity vectors) predicted for the same discharge

REFERENCES

Amsler, M. and Schreider, M. 1999, Dunes height prediction at floods in the Parana River, Argentina, in River Sedimentation, Jayawardena, A. W., Lee, J. H. W., and Wang, Z. Y., eds., Balkema, Rotterdam, pp. 615-620.

Benque, J. P., Cunge, J.A., Feuillet, J., Hauguel, A., and Holly, F. M., Jr., 1982, New method for tidal current computation, Journal of the Waterway, Port, Coastal and Ocean Division, ASCE, Vol. 108, No. WW3, pp. 396-417.

Bombardelli, F., Menendez, A., and Brea, J. D., 1997, A mathematical model for the lower Parana River delta. 3rd International Conference on River Flood Hydraulics, Stellenbosch, South Africa.

Chow, V. T., 1959, Open Channel Hydraulics, McGraw-Hill, New York.

De Saint Venant, B., 1871, Théorie du movement non-permanent des aux avec application aux crues des rivières at à l'introduction des marées dans leur lit, Comptes Rendus des Séances de l'Académie des Sciences, Vol. 73, pp. 148-154 and pp. 237-240.

Havnø, K., Madsen, M. N., and Dørge, J., 1995, MIKE 11- A generalized river modelling package, in Computer Models of Watershed Hydrology, Singh, V.P., ed., Water Resources Publications, Highlands Ranch, Colorado, pp. 733-782.

Kuipers, J. and Vreugdenhil, C. B., 1973, Calculations of two-dimensional horizontal flow. Report S163, Part I, Delft Hydraulics Laboratory, The Netherlands.

Paoli, C. and Schreider, M., eds., 2000, The Middle Reach of the Parana River: Contribution to its Knowledge and to Engineering Practices in a Large Flat-lands River. National University of the Litoral, Santa Fe, Argentina (in Spanish).

Schumm, S. A. and Winkley, B. R., 1994, The Variability of Large Alluvial Rivers, ASCE Press, New York.

U.S. Army Corps of Engineers (USACE), 1997, User's guide to RMA2-WES, Version 4.3, U.S. Army Corps of Engineers – Water Ways Experimental Station Hydraulics Laboratory, Vicksburg, Miss.

U.S. Army Corps of Engineers (USACE), 2001, HEC-RAS River Analysis System. U.S. Army Corps of Engineers, Hydrologic Engineering Center, Users Manual CPD-68, Version 3.0, January, Vicksburg, Miss.