Replacement of ponte Don Bosco: A r.c. variable thickness skew vault in the urban environment

A. De Luca and A. De Martino
University of Napoli Federico II, Department of Structural Engineering, Napoli, Italy

F. Ricciardelli
University of Reggio Calabria, Department of Mechanics and Materials, Reggio Calabria, Italy

G. Mautone
Professional Engineer, Napoli, Italy

ABSTRACT: Replacement of existing bridges located in the urban environment is in many cases a complicated task, as it requires a number of problems to be solved, not all of structural nature. These can be associated with geometric constraints and architectural requirements, but also with the interaction of the new construction with surrounding activities and other existing structures and utility networks. Therefore, in many cases the final design turns out to be a trade-off solution, coping with the issues above. In this paper the design of a 27 m reinforced concrete, variable thickness skew vault bridge is presented, presently under construction in the city of Napoli. In spite of the structure small size, a number of problems have arisen in the design and construction stages. In particular, from the structural point of view, the 35° skew vault has shown an interesting static behaviour, hard to predict through existing mechanical models.

1 INTRODUCTION

Corso Novara was conceived as one of the main avenues of the ancient urban plan of the city of Napoli, and such has remained over the centuries: its location appears in the old maps, among which the famous one by Rizzi Zannone, dated 1794. Due to the enormous increase in population density and vehicular traffic, in 1985 a viaduct (shown in figure 1) was built within the street canyon, in order to increase traffic capacity. Towards its northern end, Corso Novara intersects via Don Bosco, a street heading to the hill of Capodichino, where the International Air-

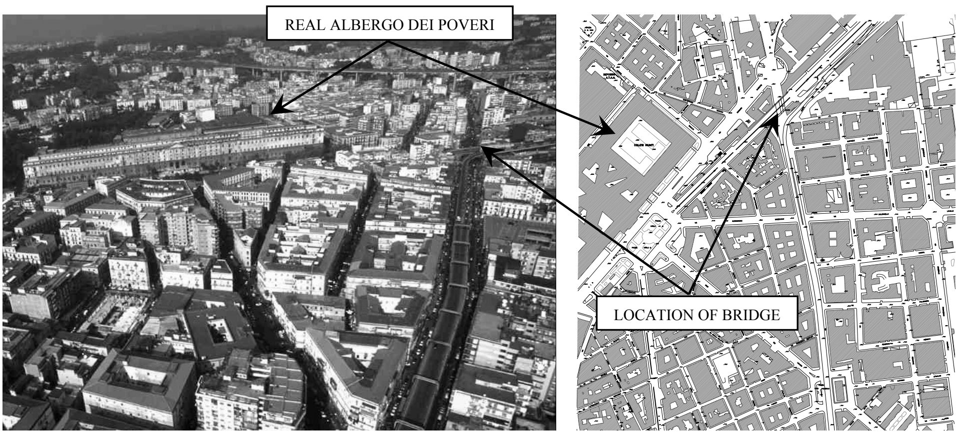

Figure 1 : Bird's eye view and plan of the area of Corso Novara and via Don Bosco

Figure 2 : Old ponte Don Bosco

port of Napoli is located. This intersection, representing the location of the bridge, is shown in figure 1, in which on the left a bird's eye view of the area is given, while on the right the plan of the same area is provided. As shown in the figure the bridge is located very close to the Real Albergo dei Poveri, one of Europe's XVIII century largest buildings. Via Don Bosco is therefore part of the road link between the Central Station and the Airport. The old ponte Don Bosco, shown in figure 2, is the assemblage of two different cylindrical skew vaults, the one on the left being a semi-circular vault, and the one on the right being a shallow vault. The types of bricks used for the two vault were also different from each other in terms of size and quality. In 1999, according to a renewal plan for the eastern area of Napoli, the Corso Novara viaduct was demolished, and the street given its original appearance. Within the same renewal plan, the Municipality of Napoli decided to replace the existing bridge with a lighter single span, allowing a higher traffic capacity. Construction started in July 2006, completion is expected to take place in summer 2007. Structural design was done by Antonello De Luca, and contractor is Fico Costruzioni. More details are given at the end of the paper.

2 CONSTRAINTS AND DESIGN OF THE BRIDGE

2.1 Constraints

Due to its location within an old and densely populated area, the design of the via Don Bosco bridge was influenced by a large number of constraints. Among these, a major role was played by existing utility networks., located as shown in figure 3a. Among the many constraints, those which most strongly affected the design of the new bridge were:

1. two 1000 mm diameter water ducts, which were incorporated in the old bridge parapets (see figure 3a); these were replaced by a bypass made of four 600 mm diameter ducts and one 1000 mm duct hosted in two hollow box-type piers, as shown in figure 3b; the room available inside the abutments was used for placing the gate valves of the water ducts;
2. a major sewer, shown in figures 3a and 3b, which made it complicated the casting of the east pier; this made it necessary the addition of a thin R.C. wall inside the sewer, and a sequential phase of casting of the foundations adjacent to the sewer;
3. a number of cables, including electrical, telecom (a bundle of 24,000) and high voltage power, which required modification of the original design, featuring a connecting slab be-

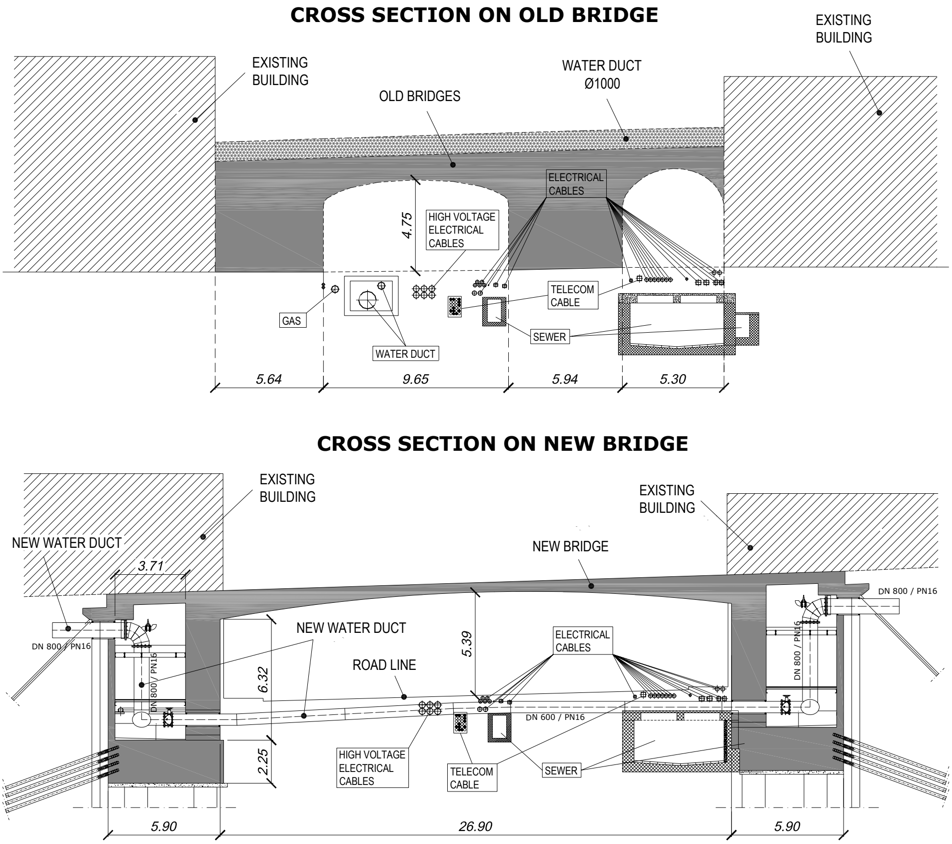

Figure 3 : Sections of the old and new bridges, showing interference with utility networks

tween the piers foundations; two to four inclined micropiles between each pair of piles of the retaining walls were added instead, as shown in figure 3b;

4. requirement of a minimum clearance of 5 m, which addressed the structural solution towards a very shallow, variable thickness vault system, as it will be better explained in the next paragraph;
5. road geometry which obliged to respect elevation points of via Don Bosco and which uniquely defined a skew plan for the new structure;
6. existing buildings which required some underfoundation interventions during construction, since foundations of the new bridge required excavation up to five metres adjacent to the buildings.

2.2 The structural solution

In order to fulfil the requirements above, the following geometric dimensions resulted for the new bridge: net span of 26.90 m, breadth of 19.85 m, and angle of skewness of 35°. All the dimensions and elevations are given in figures 4a and 4b (taken from the original drawings), which represent the sectional views respectively in th *x-z* and *x-y* planes. The bridge has two angles of inclinement, in the *x-z* and *y-z* planes, respectively.

The final project consisted in a skew reinforced concrete vault with variable thickness and inclined roadway. The minimum thickness at crown is 0.50 m, while the maximum thickness at the abutments is of 1.50 m. Figure 4b represents the construction drawing defining the geometry

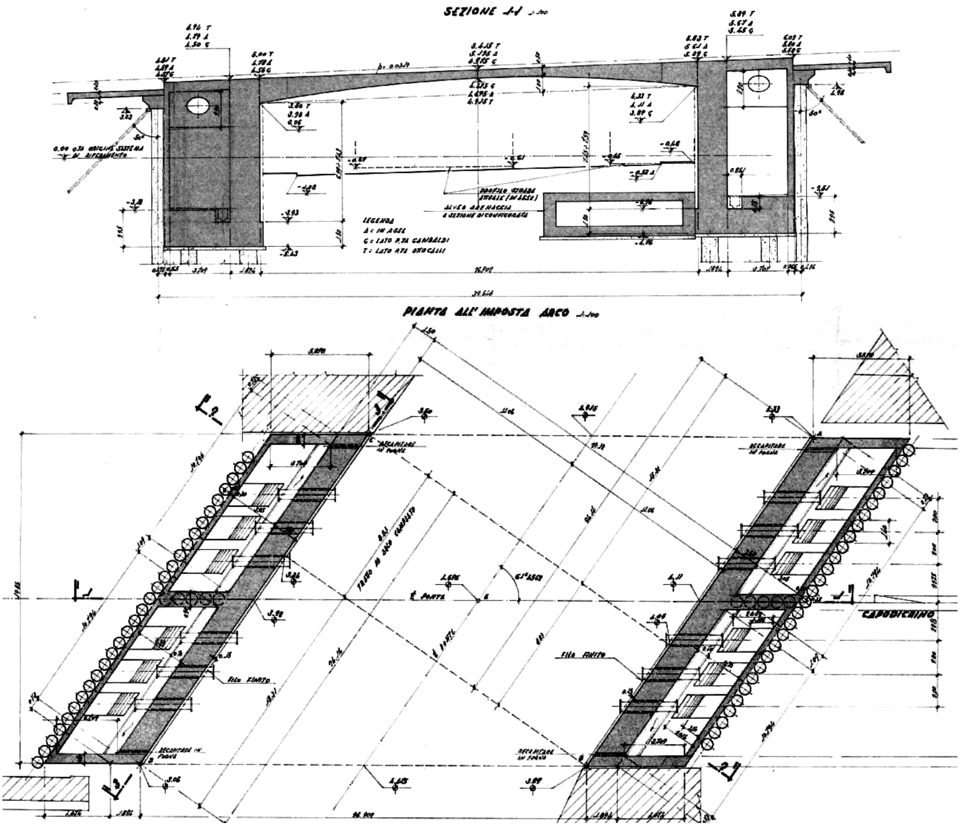

Figure 4: Longitudinal and horizontal sections from the original design drawings

of the three-dimensional structure. In this figure the geometrical centre and the axis of the bridge are represented. The axis is parallel to the two inclined abutments, and represents the line of minimum thickness of the bridge. Also in figure 4b, two inclined dashed lines, perpendicular to the abutments are given; they respectively start from the north-west and south-east corners, and define the construction cylinder, with a span of 22.13 m and with a circular radius of 61.66 m.

The abutments are made of two hollow reinforced concrete blocks, about 10 m high and 5.90 m thick. The foundations are made of 20, 1000 mm diameter piles on each side, 30 m long. This foundation was completed with a retaining wall made of 700 mm piles, with anchoring micropiles on the top. During the construction stage the foundation design was modified to include 96, 175 mm diameter micropiles at mid-height of the retaining wall. The micropiles, then, have the dual purpose of limiting the horizontal displacements of the vault abutment, and reducing the size of the connection between the foundations of the two abutments. The sectional structural view of the bridge, showing the shallow vault, the box piers capable of accommodating for the large thrusts and some reinforcements, together with the principal geometrical dimensions, is given in figure 5.

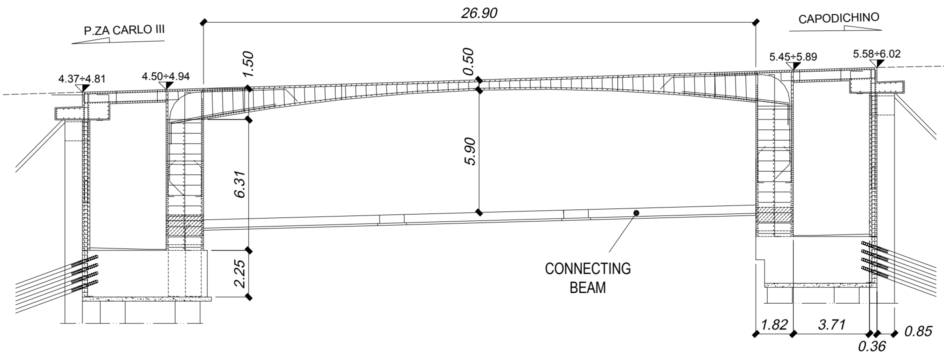

Figure 5 : Bridge section, with reinforcements and principal geometrical dimensions

3 ANALYSIS OF THE BRIDGE AND ISSUES ON STRUCTURAL BEHAVIOUR OF SKEW VARIABLE THICKNESS VAULTS

The structural behaviour of the ponte Don Bosco presents some aspects of interest since its geometric peculiarity makes it not immediate to predict neither easy to find in handbook solutions. Two sets of FEM analyses were carried out in the design stage, both using the SAP 2000 software. The first set of analyses was carried out on a simplified model of the vault, not including the abutments, the foundations or any other structural component except for the vault itself, whose results of the analyses will be briefly discussed in the following. A second set of analyses was carried out on a more detailed model of the structure, including abutments, foundations and secondary structural elements. The results obtained on the detailed model were used for the final design of the bridge, and will not be discussed here.

The simplified FEM model is made of 1720 shell elements, and has fixed external constraints, therefore does not incorporate abutment and foundation deformability. Together with the actual bridge configuration, two additional configurations were considered for the purpose of comparison. The first is a rectangular bridge having the same span, width and thickness variation as the actual one. The second is a skew vault of constant thickness, chosen such to bring the same deflection at midspan under traffic loads as the actual bridge.

In figure 6a, 6b and 6c the principal compression (forces per unit length) in the vault subjected to traffic loads are shown, for the actual bridge, for the rectangular variable thickness vault and for the skew vault with constant thickness. Figure 6b shows that the rectangular, variable thickness vault is subjected to an almost uniform compression state, in the range of 0.39 to 0.44 MN/m, with only minor edge effects. This result indicates a non optimal exploitation of the material, associated with the variable thickness of the vault. A quite different compression distribution can be seen in figure 6a for the skew, variable thickness vault, indicating the establishment of a stiff arch in the direction of the short diagonal of the vault plan. In this case the variability of the compressive force per unit length is much more pronounced, as these range from 0 to 0.9 MN/m. The existence of almost unloaded areas of the vault is accompanied with large stress concentrations at the ends of the stiff arch. In addition the distribution of the compressive force along the short diagonal is almost proportional to the vault thickness, indicating an almost uniform distribution of stresses in the material. Finally, from figure 6c it can be seen that for the skew, constant thickness vault the distribution of the compressive force follows the same pattern as in the case of the skew, variable thickness vault. In this case, however, the ratio of the maximum to minimum compressions along the short diagonal is in the order of 2, as opposed to a ratio of the vault thickness in the order of 3. This indicated a larger stress concentration with respect to figure 6a representing the variable thickness skew vault of ponte Don Bosco.

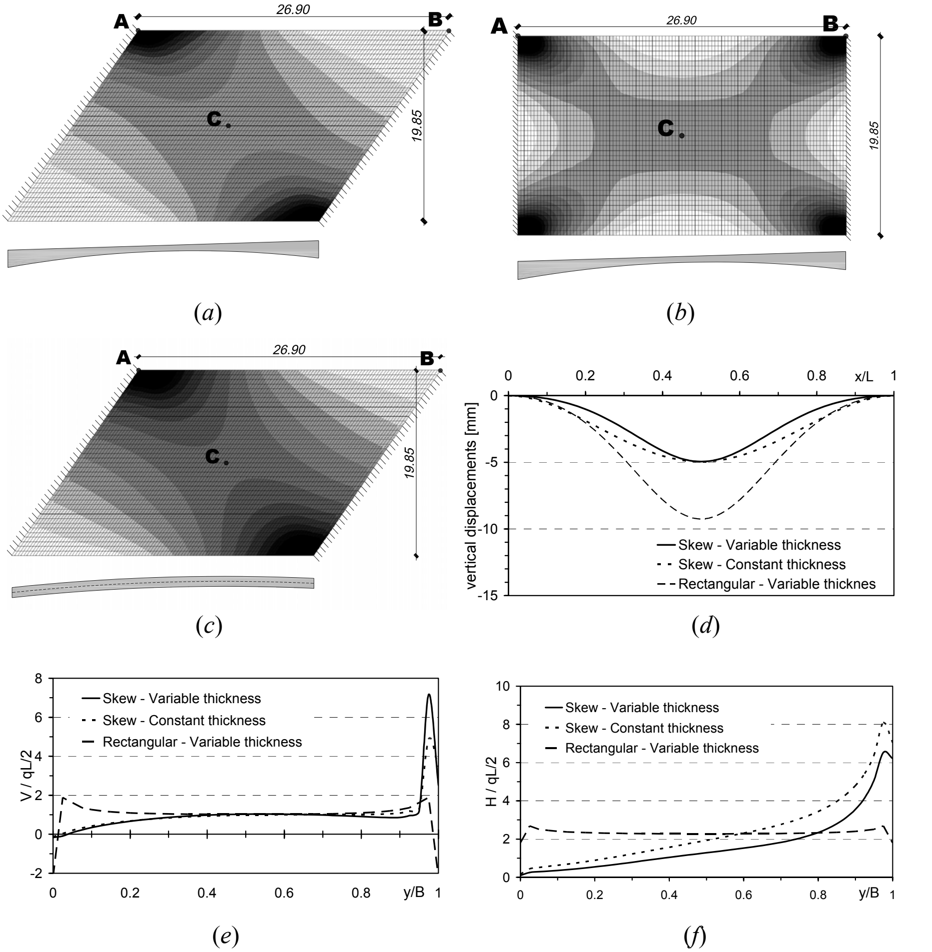

Figure 6 : Results of FEM analyses; maximum compressive stresses in the skew, variable thickness vault (*a*), in the rectangular, variable thickness vault (*b*) and in the skew, constant thickness vault (*c*); vertical displacement at midspan (*d*); vertical (*e*) and longitudinal (*f*) reaction forces.

Comparison between the three different models represented in figures 6a, 6b and 6c has proved that the structural behaviour of skew variable thickness vaults is quite different from that of rectangular vaults, as it features a pronounced three-dimensional stress distribution, associated with the establishment of a diagonal stiff-arch resisting mechanism. These findings are also confirmed by figure 6d, in which the vault deflection under traffic loads is shown for the three configurations. It appears that the skew, variable thickness vault is almost twice stiffer than the rectangular, variable thickness vault, confirming the better exploitation of the material. On the other hand, also the uneven stress distribution along the diagonal of the skew, constant thickness vault is confirmed by figure 6d. The crown deflections of the two skew vaults are the same, as this is the equivalence criterion used to set the uniform vault thickness, however, the reins deflections of the constant thickness vault are much larger than those of the variable thickness vault, as the effect of the large stresses occurring at crown.

The larger stiffness of the skew vault with respect to the rectangular vault, and therefore its better structural behaviour, is obtained at the expenses of a high concentration of reaction forces at the ends of the diagonal arch. This is shown in figures 6e and 6f, where the non-dimensional vertical and longitudinal reactions per unit length are shown. For the rectangular vault the reactions are uniformly distributed except for the edge zone where the well known uplift edge effect is visible, and the ratio of the maximum to the mean reaction never exceeds the value of 1.9. In the case of the skew vaults both the vertical and longitudinal reactions are concentrated towards the ends of the diagonal arch, with a ratio of maximum to mean reactions up to a value of 8.

The results obtained on the simplified model were confirmed by the detailed model including abutments and foundations, which also showed a pronounced 3-dimensional stress distribution in the vault. On the other hand, as only the case of the skew, variable thickness vault was considered in the detailed model, it was impossible to compare the actual behaviour with that of the rectangular and constant thickness vaults.

From the previous analyses it can be concluded that in the case of a shallow vault, the variable thickness well combines with some skewness, as they together contribute to the diagonal stiff-arch resisting mechanism, bringing at the same time a concentration of forces but a more uniform distribution of stresses. In this regard, it becomes of interest to assess in what circumstances the skewness of the vault becomes dominant, and the behaviour of the vault departs from that of a rectangular one.

4 DEFINITION OF GOVERNING PARAMETERS AND EQUATIONS

In order to obtain some insight of what observed in the previous paragraph an attempt is made in this conclusive section of the paper to define some parameters and some procedures useful for design purpose.

Within this framework it is proposed to define the parameter β for skew vaults as:

$$\beta = \frac{D}{L} \tag{1}$$

where D is the length of the diagonal and L is the span of the vault.

This parameter β, in order to account for the angle of skewness α and for the ratio γ of span (L) to width (B), can be put in the following form:

$$\beta = \frac{D}{L} = \frac{1}{\gamma}\sqrt{(\gamma - tg\alpha)^2 + 1} \tag{2}$$

Definition of the β parameter, allows to set the ranges in which the behaviour is closer to that of a rectangular vault and those in which a stiff-arch mechanism develops. In particular, values of β larger than one indicate a behaviour close to that of a rectangular vault, while values lower than one indicate a skew behaviour sensibly different from that of a rectangular vault. A confirmation of these values is given by the particular case of the Don Bosco bridge, where the value of the β parameter equal to 0.7 confirmed the dominant stiff-arch mechanism.

From eq. (2) it is possible to calculate the limit value of skewness α_l, such that for larger values the stiff-arch mechanism becomes dominant:

$$\alpha_l = \arctan\frac{1}{\gamma}\left(\gamma - \sqrt{\gamma^2 - 1}\right) \tag{3}$$

For the Don Bosco bridge α_l=18.8°, almost half the actual skewness.

It is obvious that a larger parametric analysis based on the principles stated herewith will allow to more exactly define the limit values of parameter β and of limit value of skewness angle α_l.

Based on the observations above, it is clear that the theory established for arches and rectangular vaults is not applicable for the design of skew vaults, and some more appropriate tool is necessary for the closed form analysis of such structures. For the case of skew plates it is possible to write the equilibrium equations using a skew reference frame, which has the advantage of an easier definition of the boundary conditions, and of the derivation of more meaningful skew force components. These components would be very useful also in the case of reinforced concrete structures where reinforcements are usually located in the skew directions.

Based on these considerations, it results very important to write the equilibrium equations using the skew reference frame, which for a dominant membrane behaviour will result:

$$\begin{cases} \dfrac{\partial N_{\bar{x}}}{\partial \bar{x}} + \cos\alpha \dfrac{\partial N_{\bar{x}y}}{\partial y} + \sin\alpha \dfrac{\partial N_y}{\partial y} = 0 \\ \sin\alpha \dfrac{\partial N_{\bar{x}}}{\partial \bar{x}} + \sin\alpha \dfrac{\partial N_{\bar{x}y}}{\partial y} + \dfrac{\partial N_y}{\partial y} = 0 \\ N_y = R\cos\alpha Q \end{cases} \tag{4}$$

where $N_{\bar{x}}$ and N_y are the compression axial forces per unit length in the longitudinal (skew) and lateral directions, and $N_{\bar{x}y}$ is the coupling term. R is the radius of curvature of the vault and Q the vertical load per unit surface. Eqs. (3) are similar to those applying for rectangular vaults and can be solved following the same approach.

$$\begin{cases} N_{\bar{x}} = f(y) \\ N_{\bar{x}y} = g(\bar{x}) \\ N_y = R\cos\alpha Q = const. \end{cases} \tag{5}$$

where $f(y)$ and $g(\bar{x})$ are functions depending only on the boundary conditions.

5 CONCLUSIVE REMARKS

In this paper the replacement of ponte Don Bosco has been presented. The implications in design of the numerous constraints deriving from the densely populated urban environment have been addressed, with particular attention to the interference with the utility network. The solution adopted of a shallow variable thickness skew vault was investigated by comparing the stress distribution of the actual structure to other more straightforward and common structures: rectangular variable thickness vault and constant thickness skew vault. The particular diagonal stiff-arch resisting mechanism was adequately commented and a parameter which should indicate arisement of this stiff-arch resisting mechanism has been defined for design applications.

MAIN DATA OF THE PROJECT AND A CONSTRUCTION PHASE OF THE WEST PIER

Client/Owner	Comune di Napoli
Responsible	Arch. Giuseppe Pulli
Contractor	A.T.I. – Fico Costruzioni s.r.l. Amato trivellazioni s.r.l. Fico Giuseppe
Structural design	Prof. Ing. Antonello De Luca
Foundations	Prof. Ing. Carlo Viggiani