# 12. SEISMIC ASSESSMENT OF BRIDGES IN THE CATANIA AREA

## 12.1 Introduction

The importance of lifeline survival has gathered increasing attention in the past few years, after damaging earthquakes which struck densely urbanised areas. In particular, the serviceability of roads is a critical issue, since after a strong earthquake it is necessary to transport large amounts of goods and persons in a very short time period.

In this light, the purpose of carrying out a seismic assessment of existing bridges is to determine the level of risk associated with loss of serviceability, severe damage, or collapse. Once this risk has been quantified, rational decisions can be taken as to whether the bridge should be retrofitted or replaced, or to accept the risk and leave the bridge in the existing state. There are generally two stages to a seismic assessment. The first involves a general screening and prioritisation study (see section 7), to determine which bridges are most likely to pose the greatest risk. This is normally carried out based on generic indicators, such as age of bridge, soil conditions, structural type, local seismicity and traffic density, rather than on detailed analyses. The second stage involves a detailed structural analysis of the bridges identified in the prioritisation phase as having high risk in relation to local seismicity and soil conditions. This stage may adopt either a "pass-fail" point of view to evaluate the bridge survival relative to some pre-determined limit state, or, be oriented to defining the bridge position in a continuous damage space. Both approaches may be deterministic or probabilistic; in the latter case either the statistics obtained from previous earthquake are directly available for the quantity at hand, or it is possible to derive them from functional relations with parameters whose statistics are known. These functional relations are usually obtained from mechanical models of the structure.

In the following, a deterministic approach to assess the survival of three different bridges to the scenario earthquake is described (12.2), together with a probabilistic approach (applied to all the selected bridges of section 8) which allows to assess the probability of exceeding some damage levels (12.4). The two methods give quite consistent results, indicating that the bridges considered are substantially safe even under the level I scenario earthquake. In (12.3) the "Strada Provinciale 69" overcrossing of the Catania "*Tangenziale*" has been studied in detail. Unlike the methods of (12.2) and (12.4), which resort to simplified structural models, a detailed model of the overcrossing has been used, including soil-structure interaction effects.

## 12.2 Seismic assessment of bridge piers

*(G.M. Calvi and A. Pavese)*

This contribution deals with on the seismic assessment of some case studies of bridges, located in Catania; the assessment is performed by means of a non linear simplified analysis. After a brief review of the models used, the response in terms of moment-curvature and force-displacement curves of some cases are presented.

### 12.2.1 *Modelling and analysis*

The seismic response of the bridges can in general be determined using a non linear analysis, where all possible sources of non linearity should be considered. In this context, there is a wide choice of models which correspond to different levels of accuracy in the representation of the phenomena. For the assessment of a large number of structures, as in the case of Catania, the push-over analysis combined with simplified models appears to be the most acceptable solution both for the level of accuracy and the relatively limited time required for the calculation. In the present work, a restricted number of bridges (Figures 12.1 to 12.3) have been selected from the whole population of such structures in Catania and the seismic response has been calculated with the computer code SVVS (Calvi *et al.* 1997) implemented by the authors. In the following paragraph the basic models implemented in the code are briefly presented.

#### 12.2.1.1 *flexural strength*

The flexural strength of the pier has been calculated using a fiber-model implemented in SVVS. To obtain the best estimation of the flexural strength and the deformation capacity, a moment-curvature analysis incorporating effects of confinement of the concrete core by transverse reinforcement and strain hardening of longitudinal reinforcement has been used. The Mander (Mander *et al.*, 1988) stress-strain relationship for both confined and unconfined concrete regions of the section and the experimental curve for FeB44 k steel have been adopted in the analysis. The model includes also the effect of strain hardening penetration in the foundation by extending the plastic hinge length of 1/10 of the pier height and the p-δ effect. The computed moment-curvature curves are illustrated in Figure 12.4.

The relations between the base shear and the lateral deflection of the pier have been determined by integrating the curvature distribution over the pier height, and the effects of the shear deformation have been also included.

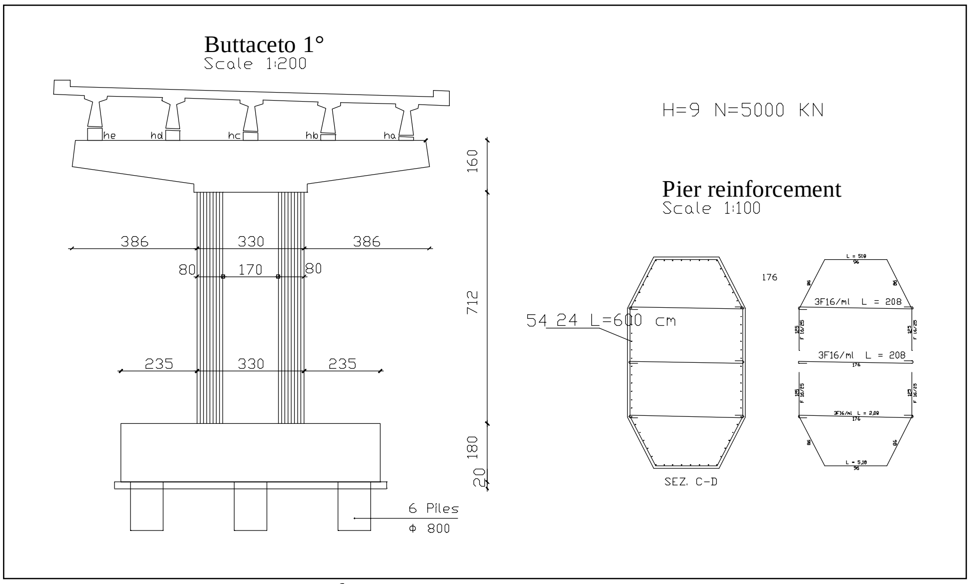


Figure 12.1: Geometric properties of Buttaceto-1

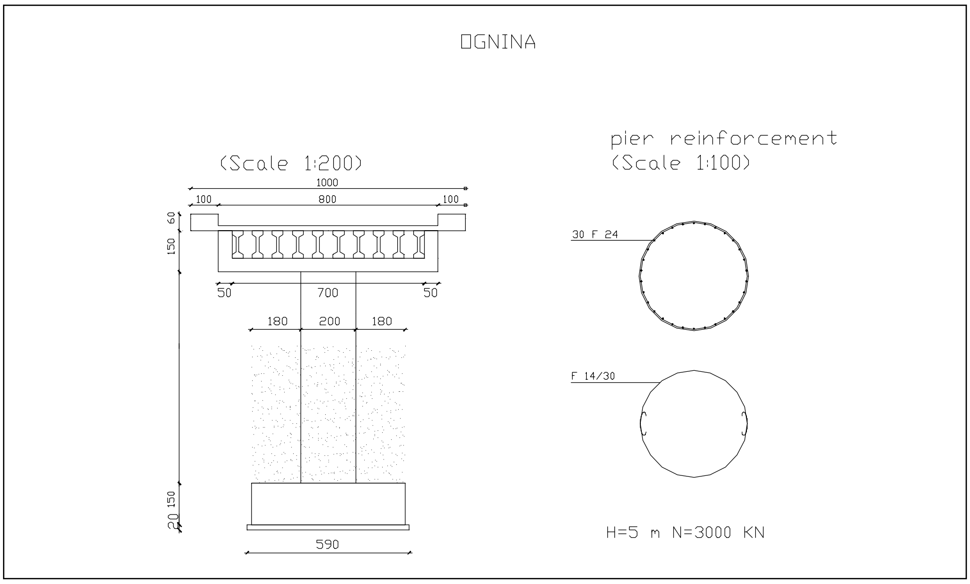


*Figure 12.2: Geometric properties of Ognina*

## 12.2.1.2 *lap spliced*

In the cases of lap-splice the failure due to loss of bonding starts when the force transmitted across the spliced bars reaches the strength capacity of the connection, which corresponds to the moment $M_n$ in Figure 12.5. After the maximum moment has been reached, both strength and stiffness degrade linearly until the residual resistance develops.

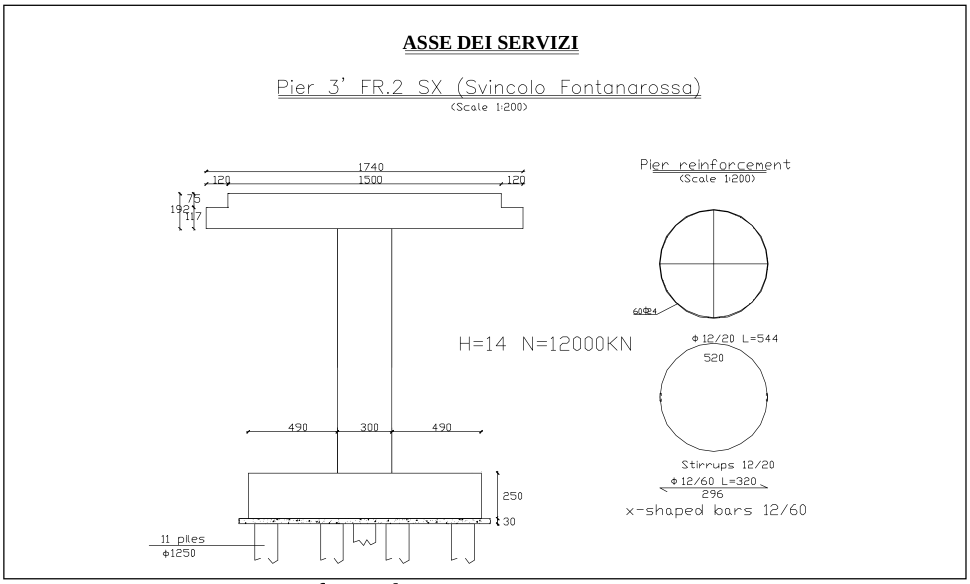

Figure 12.3: Geometric properties of Asse dei servizi

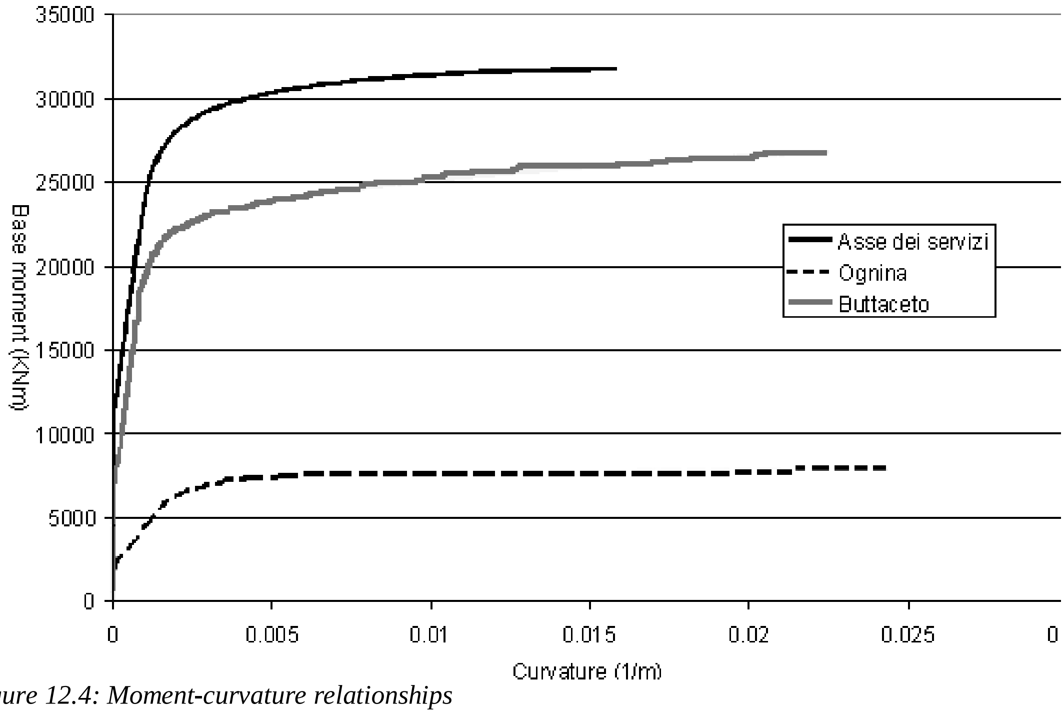

*Figure 12.4: Moment-curvature relationships*

12.2.1.3 *shear resistance*

A recent formulation proposed by Priestley et al. seems to provide a good estimate of the shear strength and the its degradation. The formulation has been recently revised to extend the prediction capability to hollow sections which are very common in bridges, especially in those with high piers. According to the proposed model, the shear resistance is expressed as the sum of the shear strengths of concrete ($V_c$), transversal reinforcement ($V_s$) and axial load ($V_p$); that is

$$V_d=V_c+V_s+V_p$$

where:

$$V_c = \alpha\beta\gamma\sqrt{f_c^{'}}A_e \qquad \alpha = 3-\frac{L}{d} \qquad \beta = 0.5+20\frac{A_{st}}{A_g} \qquad \gamma = \begin{cases} 0.25 & \mu_\phi < 3 \\ 0.25-\dfrac{0.2}{12}\mu_\phi & 3 \le \mu_\phi \le 15 \\ 0.05 & \mu_\phi > 15 \end{cases}$$

$$V_s = \frac{A_v f_{yh}(h-c-c_0)ctg\,35}{s}$$

$$Vp = 0.85\,P\left(\frac{h}{2}-\frac{c}{2}\right)\Big/L$$

In the previous expression $L$ is the pier height, $d$ is the section depth, $A_{st}$ the cross area of the longitudinal bars, $A_g$ is the gross cross sectional area of the concrete, $c$ is the depth of the compression field, $c_0$ is the cover, $A_e$ is the effective shear area of concrete, $A_v$ is the cross area of the transversal reinforcement, $s$ is the distance between the stirrups and $\mu_\phi$ is the curvature ductility.

### 12.2.3 *Assessment results*

The analyses performed on the three selected cases show that the piers have a seismic behavior dominated by the flexural strength, the shear strength being higher, as shown in figure 12.6; this allows to conclude that the analyzed bridges have good ductility resources.

In Table 12.1 are reported the calculated PGA both for damage limit state which corresponds to the yielding of the first longitudinal bars and for ultimate limit states.

It can be concluded that only minor damage is likely to occur in the considered bridgepiers under the level I scenario earthquake.

Table 12.1: PGA values (in g) corresponding to Damage and Ultimate Limit States

| Bridge | Period (Sec) | PGA DLS | PGA ULS |
|---|---|---|---|
| Buttaceto 1° | 0.41 | 0.33 | 3.19 |
| Ognina | 0.67 | 0.28 | 1.68 |
| Asse dei servizi | 1.21 | 0.28 | 6.88 |

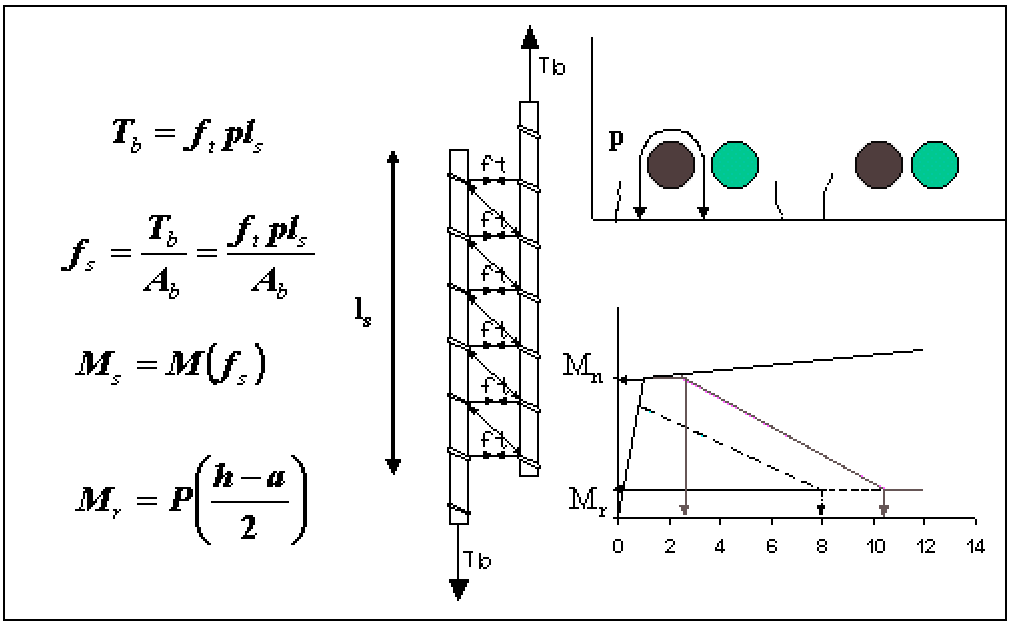


*Figure 12.5: Model for seismic assessment of columns with lap spliced longitudinal reinforcement*

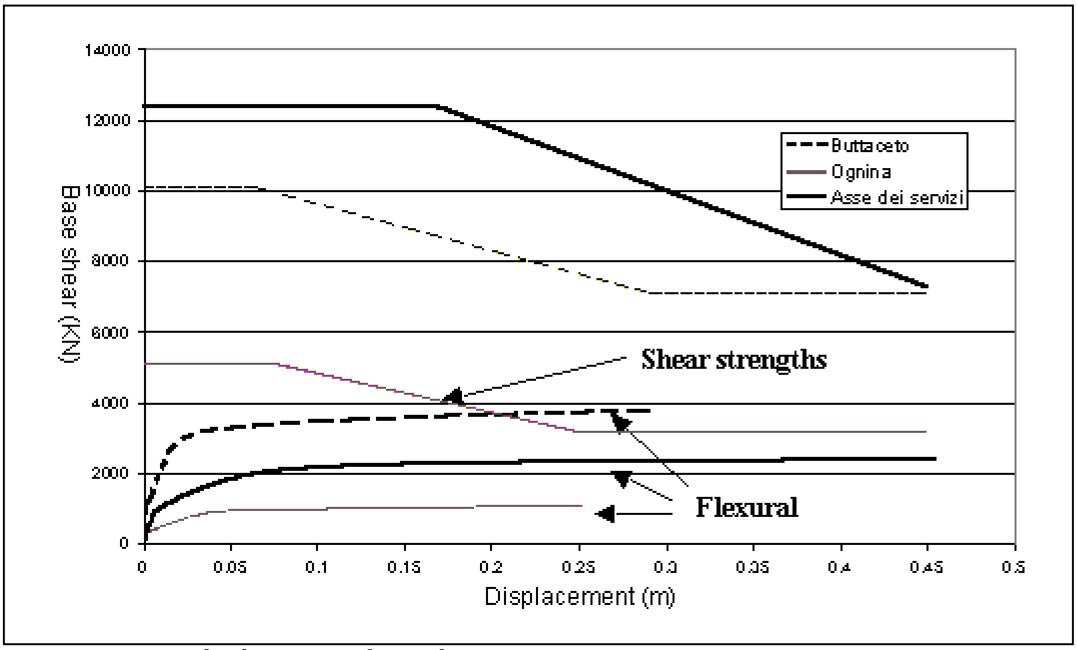


*Figure 12.6: Force-displacement relationships*

## 12.3 Detailed analysis of a typical overcrossing

*(L. Martinelli and F. Perotti)*

As a part of the scenario study of bridge structures, the "Strada Provinciale 69" overcrossing the "*Tangenziale*" expressway has been carefully studied. For type of construction, pier shape and span length, the overcrossing in question can be considered typical for the area.

A detailed model of the overcrossing, including soil-structure interaction effects, has been studied by means of a modified version of the well-know SAPIV software (Bathe *et al.*, 1973). Using both response spectrum and time history analyses, the peak ground acceleration (PGA) corresponding to collapse has been computed for the scenario ground motion. In particular, it has been possible to estimate the PGA corresponding to collapse of the most critical structural elements. Analyses with a non linear pier model (Martinelli, 1999) are still in progress.

### 12.3.1 *Modelling of the overcrossing*

The overcrossing (whose location is shown in Figure 7.1) is a three span, simply supported, structure (see Figures 12.7 and 12.8); the end spans length is 12.7 m while the middle one is 31.7 m. Both the abutment walls and the piers are founded on 20 m long piles having a cross-section diameter of 1.0 m. The superstructure is made of precast, prestressed "T" beams with a cast-in-place connecting slab; the beams rest on elastomeric bearings at the abutments and at the piers. The deck longitudinal axis is approximately oriented West-East.

To properly account for both horizontal and vertical motion of the superstructure, several models have been tested; the one adopted in the analysis allows for a significant reduction of degrees of freedom without loss in representing modal shapes and frequencies of interest. The superstructure weight is 1872 Kg/m; to simulate out-of-plane behaviour the deck has been modelled through a grillage composed of longitudinal beams, one for each prestressed beam, and transverse connecting beams spaced at 5.35 m for the central span and 3.27 m for the outward ones. At these locations lumped masses were considered. In-plane stiffness is treated assuming the plane section hypothesis and introducing a single longitudinal beam (having the cross-section of the deck slab) whose centroid is rigidly connected to the prestressed beam axes.

The pier, see figure 12.7, is of the solid cross-section type having dimensions 5.78x0.80 meters at the base; it is reinforced with 32 bars of 24 mm diameter, 13 along the long sides and 3 intermediate along the short sides. At the footing, a 1.5 m overlap length with starter bars is present. Transverse reinforcement is made of 14 mm ties at 300 mm spacing; ties are composed of two C shaped bars overlapping along the column cross-section short side; this may pose serious problems in case of concrete cover spalling. The pier shape is common for other overcrossing in the Catania area.

In the design drawings no information is provided on the elastomeric bearings apart for the general dimensions (15x60x4 cm), thus their performance is based on re-design results. It has been assumed the presence of six 2 mm steel layers and a Shore A3 rubber having a shear elastic modulus G=0.9 MPa.

As often recognised, inclusion of foundation modelling (though simplified), in the seismic bridge response analysis, is important. Near the structure site results from two boreholes, n. 1298 and n. 1299 in the geotechnical database( Faccioli, 1997), were available; they have been used to characterise the foundation soil. The abutment footing has been modelled as a rigid body while for the piers footing this hypothesis has been retained for horizontal displacements and rotation around the vertical axis. As suggested by geometry and the low reinforcement ratio of the footing, for vertical displacements and rotations around horizontal axis the pier footing has been considered split in the middle between the two runways. In order to capture the group effect three different pile groups, supporting the footings, have been considered: one for the abutment and two for the pier footing depending on the degree of freedom (a 16-pile group for horizontal translations and torsion and two 8-pile groups for vertical translation and rocking). Pile groups are simulated using equivalent springs having dynamic stiffness computed following Howell (1985), and adopting a quadratic variation of the soil shear modulus varying from $G$=32MPa at the pile top to $G$=250 MPa at pile bottom.

### 12.3.2 *Analysis*

Two different types of structural analysis have been performed: 3-D response spectrum and 3-D time histories. For the response spectrum analysis, the EC8 spectrum for "B" type soil has been used as well as the spectra for the level I scenario earthquake; these were computed at the overcrossing site and supplied by Priolo (see sub-section 4.2 and Priolo, 1999). Acceleration time histories corresponding to the latter have been used in time domain analysis. To broaden the applicability of the results, analyses have been run for two different structural situations: the first one (model F in the following) assumes fully functional bearings in the horizontal plane. The second one (model R) assumes the central span restrained in the horizontal plane at one of the piers and free at the other in the bridge direction and the outer spans similarly restrained at the back walls and free at the piers; this configuration simulates the introduction of shear keys. Seismic internal forces and relative, as well as absolute displacements, have been obtained from the analysis.

### 12.3.3 *Assessment*

Structural collapse can be induced by lack of resistance in any ring of the resistance chain: foundations piles, pier shear resistance, pier flexural resistance, anchorage of flexural reinforcing bars, bearings deformation, stability or sliding, etc. Relative to these causes, the overcrossing seismic resistance has been assessed for the scenario earthquake and, in addition, for the EC8 design spectrum. Resistance and local ductility of structural members have been assessed using both a conservative estimate of nominal concrete and steel mechanical properties ($f_c$=25 MPa, $f_{sy}$=430 MPa), and a set of values closer to the measured material resistance ($f_c$=50 MPa, $f_{sy}$=450 MPa, see chapter 8), according to both EC8 and the procedures reported in (Priestley *et al.*, 1996). Subsequently, on the basis of local ductility, the structural displacement ductility has been computed relative to the different collapse causes to be assessed. Using this global ductility, the collapse acceleration, represented by the earthquake PGA, has been computed. In the process, only the spectral shape of the earthquake is thus relevant. A detailed illustration of the computations performed will be presented in (Martinelli *et al.*). PGAs corresponding to failure of the foundation system have been computed using the EC8 spectrum. Failure has been identified as the inability of the pier's foundation pile group to carry any additional lateral load, thus reaching its ultimate lateral load bearing capacity, and as the failure of the most loaded pile in the pile group. Pile group failure is first reached for a PGA=1.82 m/s$^2$ while failure of most loaded pile of the pile group is reached for a PGA=0.77 m/s$^2$ for "model F" and 0.40 m/s$^2$ for "model R" respectively. Using the same spectrum, collapse for excess of shear deformation in the deck bearings is reached at a PGA=0.69 m/s$^2$ while sliding of the bearing on the pile cap (assuming no fastening device is present) is possible for PGA>0.50 m/s$^2$.

Assuming equal PGA, it is to be noted that internal actions due to the "scenario" earthquake spectra are about 90% of that due to the EC8 response spectrum; a slight increase in PGA foundation collapse values are, thus, to be expected.

As it regards the pier, the critical section appears to be the base section. Collapse might be of brittle type, due to collapse in shear before full flexural plasticization of the pier's base cross-section, or of a more ductile type if sufficient shear resistance is present to allow for such a plasticization. Shear resistance is partially due to the so called "concrete contribution" which is related to the cross-section area and to concrete compressive resistance. Concrete contribution to shear resistance is known to degrade as the experienced curvature ductility increaseas. If concrete contribution is considered, shear resistance is well enough to cause full flexural plasticization of the pier's base cross-section regardless of considered concrete resistance and concrete, while disregarding concrete contribution pier collapses in shear at a PGA= 0.33 m/s2. As reported in (Priestely *et al.*, 1996), reinforcing bars anchorage in cover concrete begins to degrade for a compression deformation $\varepsilon_c$=0.002. The PGAs corresponding to the attainment of this limit are around 4.0 m/s$^2$ and 8.0 m/s$^2$ for "model R" and "model F" respectively, using a concrete resistance $f_c$=25 MPa. PGAs corresponding to confined concrete reaching the deformation $\varepsilon_c$=0.005 are almost double. At this PGA level flexural steel is still well below the deformation limit ($\varepsilon_s$=0.10) corresponding to a probable bar collapse. Collapse PGA values show a 30% increase when computed using a concrete resistance $f_c$=50 MPa. As a last point, time history analysis has shown a relative displacement in the bridge direction, measured between the lateral bents and the middle one, in excess of 15 cm, well above the design 2 cm dilatation gap. Finally, contact among the bents is reached at a PGA of 0.71 m/s$^2$.

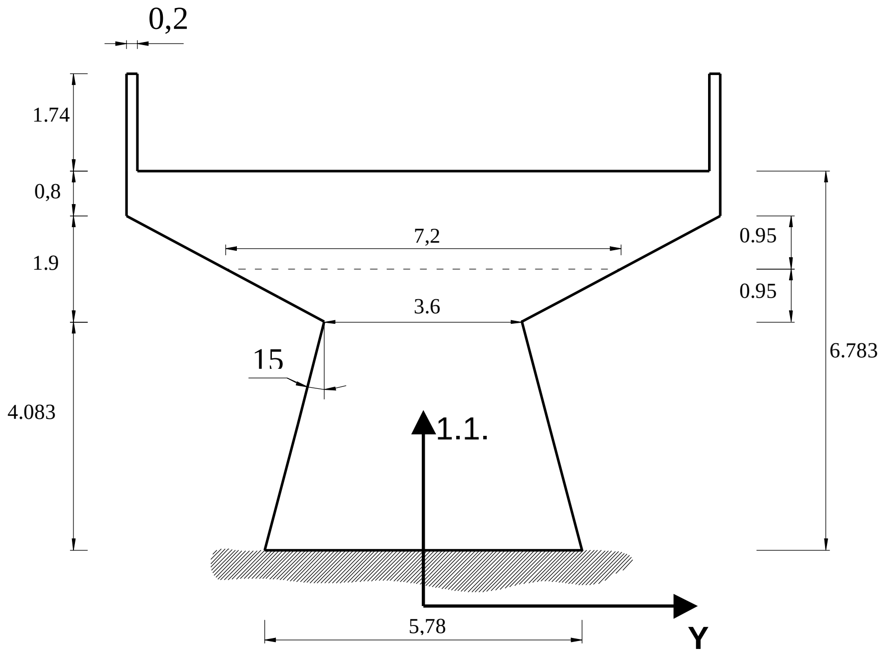


*Figure 12.7 : S.P. 69 overcrossing pier*

In view of the partial results so far obtained, the overall superstructure performance is deemed as satisfactory, but the same is not true for the bearings. On the basis of the unsatisfactory results obtained, the foundation system deserves more detailed investigations.

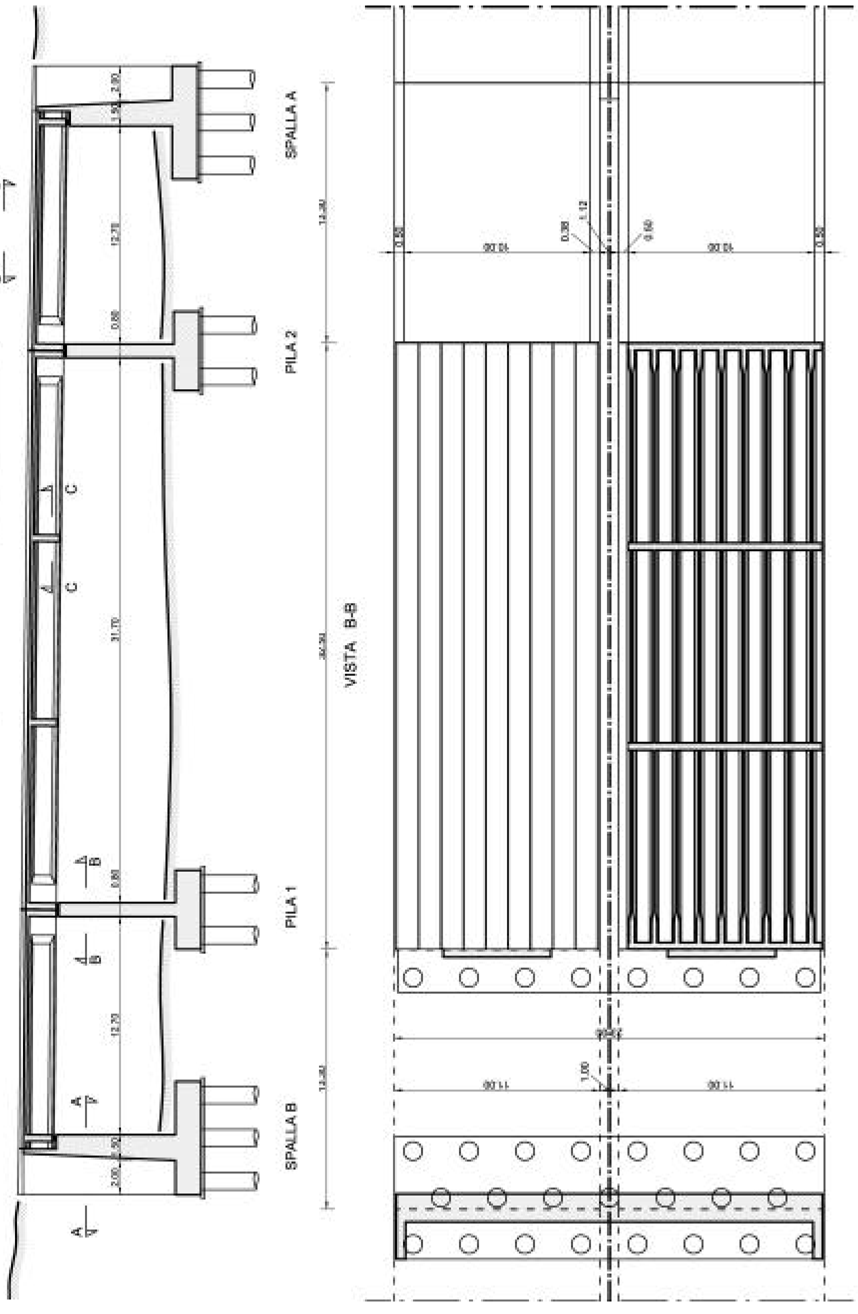


*Fig. 12.8 - Overcrossing SP69.*

## 12.4 Damage scenario

*(N. Nisticò, G. Monti)*

A damage scenario relative to the bridges in the Catania area has been developed and is presented herein with a brief description of the implementation of a Geographical Information System finalized to bridge data acquisition and elaboration.

Damage scenarios of structures like buildings or bridges can be defined either through a statistical approach or through mechanical models. If damage data are not available, only the definition obtained through a mechanical model can be used. Damage data for bridges have not been systematically collected in Italy and, consequently, Damage Probability Matrices or Fragility Functions have not been developed. Due to these considerations, the definition of damage scenarios requires the formulation of a damage mechanical model, the result of which could be checked *a posteriori* by means of experimental observations.

### 12.4.1 *Damage function*

The methodology employed to define the damage scenario is briefly outlined here; a detailed description can be found elsewhere (Monti et al., 1999).

A displacement-based damage function D is defined as follows:

$$D = \frac{s - s_y}{s_u - s_y} \quad (\text{for } s > s_y) \qquad \text{Eq.(1)}$$

where $s_y$ = structural displacement corresponding to a pre-defined yield state, $s_u$ = structural displacement corresponding to a pre-defined collapse state, and $s$ = generic structural displacement.

It is preferable to express damage in terms of acceleration rather than displacement, because ground motion is usually described in terms of PGA (Peak Ground Acceleration).

By transforming displacements into accelerations, Eq. (1) becomes:

$$D = \frac{a_y\, r(T_y, \eta(a)) - a_y}{a_y\, r(T_y, \eta_u(a_u)) - a_y} \qquad \text{Eq.(2)}$$

where:

$r(\cdot)$ = the response modification factor (Miranda and Bertero, 1994) as a function of both the conventional elastic period $T_y$ and the ductility level $\eta$ corresponding to the acceleration level $a$,

$a_y$ = PGA corresponding to the attainment of a pre-defined yield state,

$a_u$ = PGA corresponding to the attainment of a pre-defined ultimate state.

Note that damage does not depend on the acceleration producing the yield state. In Eq. (2) the dependence of the modification factor on the acceleration level *a* has been evidenced.
One can write:

$$a_y\, r\left(T_y, \eta_u\left(a_u\right)\right) = a_u\,, \qquad \text{Eq.(3)}$$

while the nonlinear expression at the numerator can be simplified as:

$$a_y\, r\left(T_y, \eta(a)\right) \approx a_y\, \eta(a) = a\,. \qquad \text{Eq.(4)}$$

Thus, Eq. (2) simplifies to:

$$D = \frac{a - a_y}{a_u - a_y}\,. \qquad \text{Eq.(5)}$$

The yield acceleration $a_y$ and the collapse acceleration $a_u$ are given as:

$$a_y = \frac{F_y}{m\, R\left(T_y\right)} \qquad \text{Eq.(6a)}$$

$$a_u = a_y\; r\left(T_y, \eta_u\right) = \frac{F_y\; r\left(T_y, \eta_u\right)}{m\; R\left(T_y\right)} \qquad \text{Eq.(6b)}$$

where *m* is the bridge mass (pertaining deck mass + pier cap mass + consistent pier mass), $F_y$ is the conventional resisting lateral force, and $R\left(T_y\right)$ is the acceleration spectrum.

Equation (2) is the adopted vulnerability function, whose values range between 0 and 1, the value 0 corresponding to the undamaged bridge and the value 1 corresponding to an ultimate state. The vulnerability function as expressed in terms of accelerations is consistent with the linear deterministic functions used to evaluate the damage scenario for buildings (see sect.12).

The determination of the mechanical parameters of equations (6) is based on the response of a mechanical model, as outlined in the following section.

### 12.4.2 *Mechanical model*

The mechanical model of the bridge is obtained by reducing the structure to an equivalent single degree of freedom system, which is then subjected to a nonlinear push-over analysis; the final result is a force-displacement relationship from which the mechanical parameters of equations (6), *i.e.* $F_y$ and $\eta_u$, are identified. The flexural behaviour obtained in this way does not account for the continuous interaction with shear. This is only considered *a posteriori* by comparing the resisting (flexural-based) force to the resisting shear force, evaluated, as a function of the required ductility, through the model proposed in (Priestley *et al.*, 1996).

The mass of the deck associated to each pier has been evaluated considering that the supports are horizontally ineffective, and therefore they behave as hinges rather than rollers. No soil-structure interaction has been considered.

#### 12.4.2.1 *Flexural strength and ductility at ultimate*

Concrete is modeled with the constitutive law proposed by Kent and Park (1971), where the ultimate compressive strain is defined as a function of the transverse reinforcement. Steel reinforcement is modeled with the law proposed in Menegotto and Pinto (1973); buckling of bars (Monti and Nuti, 1992) has been considered. For irregularly shaped columns, the lateral response is modeled with fiber-section beam-column elements; the analyses have been carried out with a general purpose, finite element nonlinear program (FEAP). For regular shape columns a simplified model has been adopted, whose accuracy has been successfully tested against the fiber element model. Based on the previous models the flexural behavior is defined by an idealized elasto-plastic force-displacement relationship, characterized by the elastic stiffness ($k_y$), the ultimate displacement ($s_u$) and the resisting force ($F_y$). The elastic stiffness is approximated by the secant stiffness at the theoretical yield point (which coincides with the first yield of a bar). The ultimate displacement corresponds either to the attainment of an ultimate limit state for concrete or steel, or to a conventional drift corresponding to the unseating of the superstructure. The ultimate load is evaluated on the basis of an energy equivalence between the idealized bilinear model and the numerically obtained force-displacement relationship.

#### 12.4.2.2 *Shear strength*

The ultimate strength is assessed using an additive equation similar to equation (1). Since the concrete contribution $V_c$ decreases with increasing ductility demand, a brittle behavior or an element ductility reduction is possible.

### 12.4.3 *Evaluation of the fragility curves*

If the system variables are deterministic, the damage function (Eq. 5) defines the damage corresponding to an expected PGA value; note that when average values are adopted for the model parameters, the resulting damage is also an average value. In a preliminary step, a deterministic damage function (vulnerability) has been evaluated: the input parameters (mass, pier height and geometry, materials parameters) have been characterized by their average values. Knowing for each bridge site the expected PGA, the expected damage has been evaluated for each pier.

Subsequent to the preliminary deterministic screening, a further probabilistic assessment has been carried out, by evaluating the fragility curves of all bridges.

In these analyses, the strength of the materials (concrete and steel) and the height of the piers have been selected as basic random variables, while the other variables have been considered as deterministic. Each random variable has been described through a Beta probability density function (PDF).

Corresponding to selected values of the PGA, the resulting damage distribution in each pier has been modeled through a Beta PDF. Subsequently, the fragility curves of each pier have been evaluated, corresponding to three selected Performance Levels (PL). The PL have been identified in terms of the ratio $a/a_u$ between the expected PGA and the PGA producing the ultimate state. The threshold between the PL-1 and the PL-2 has been chosen as $a/a_u = 0.5$, while the threshold between the PL-2 and the PL-3 has been chosen as $a/a_u = 0.7$. Collapse is identified by $a/a_u = 1$. The value of the damage index corresponding to the PL considered, depends on the available ductility level, as follows

$$D = \frac{a/a_u \cdot r(T_y, \eta_u) - 1}{r(T_y, \eta_u) - 1} \qquad \text{Eq.(7)}$$

In table 12.2, the average value of the accepted damage corresponding to the two thresholds between the selected PLs is reported as function of the available ductility $\eta_u$.

Table 12.2: Definition of the Performance Level

| | Damage D associated to the PL | | | |
|---|---|---|---|---|
| | $\eta_u = 2$ | $\eta_u = 3$ | $\eta_u = 4$ | $\eta_u \geq 4$ |
| **PL 1 – PL 2 ($a/a_u = 0.5$)** | 0.00 | 0.25 | 0.30 | 0.35 |
| **PL 2 – PL 3 ($a/a_u = 0.7$)** | 0.40 | 0.50 | 0.55 | 0.60 |

### 12.4.4 *Damage scenario*

Following the procedure previously described, the fragility curves that yield the exceedance probability of the three defined Performance Levels, have been constructed for the following bridges: an *Overpass* (or overcrossing, the most vulnerable) on the *Tangenziale*, the *Fontanarossa* bridge on the *Asse dei Servizi*, the *Ognina* bridge on the *Circonvallazione*, the *Buttaceto* bridge. For the location and significance of these lifelines, see sub-sect. 7.2. Based on the foregoing methodology and with regard to the three defined performance levels, the fragility curves of the bridges have been evaluated. Such fragility curves, illustrated in Fig. 12.9, allow to conclude that the capacity of the selected bridges is sufficient with respect to the given earthquake scenario (level I). As a matter of fact, with regard to the three PL's we have:

PL1: for *Asse*, *Ognina* and *Buttaceto* the scenario PGA (0.35g) is less than the lower bound PGA value associated to a nonzero exceedance probabilty. For the Overpass, whose associated importance category is low (see sect. 7) and the expected PGA is 0.25g, the exceedance probability is nonzero, but it is acceptable since functional collapse is allowed. For this overpass, see also the indipendent assessment in sub-sect.12.3

PL2 and PL3: for all selected bridges the scenario PGA (0.35g) is less than the lower bound PGA value associated to a nonzero exceedance probabilty.

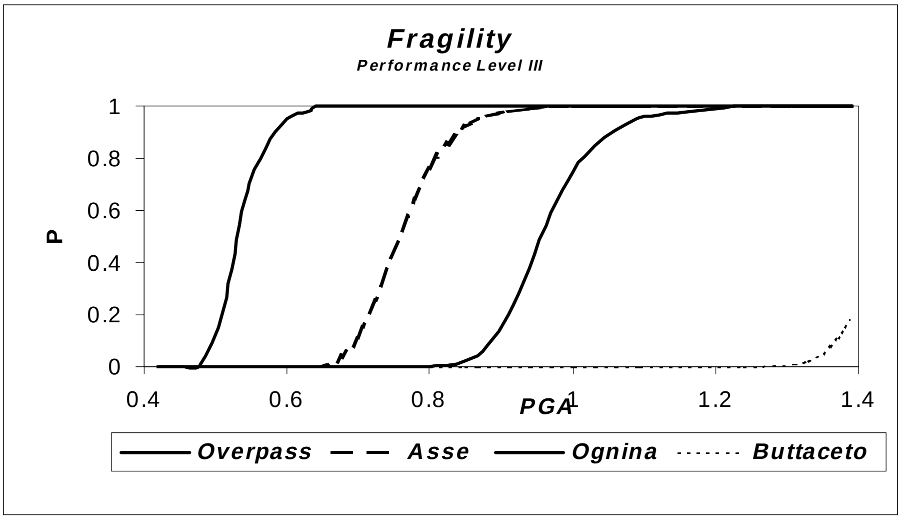


*Fig. 12.9 - Fragility curves for the selected bridges*

The present damage scenario indications at PL1 for the Asse, Ognina and Buttaceto bridges are quite consistent with the results of the seismic assessment for the damage limit state performed with a different method on the same structures, describes in sub-sect. 12.2.

### 12.4.5 *Data management*

The role of GIS (Geographical Information Systems) in the definition of damage scenarios is nowadays widely recognized. To asses the structural behavior of both buildings and bridges, the use of systems that integrate GIS, CAD system and structural computer code appear a good way to optimize the work. Within the framework of defining seismic scenarios for bridges, a preliminary release of such a GIS has been implemented in a CAD environment, integrating a pre-processor that allows to locate the bridge on a topographic map, to store and visualize the bridge structure data, and to assess the structural behavior. The system integrates Autocad, Access, and FEAP as well as simple routines to assess the bridges.

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