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STRUCTURAL ANALYSIS AND FATIGUE RELIABILITY ASSESSMENT OF THE PADERNO BRIDGE

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ABSTRACT: The Paderno Bridge along the Adda river in the northern part of Italy is a relevant arch-bridge connecting Milan and Bergamo province in the northern part of Italy: the bridge is part of the Monza-Bergamo line and is in service from 1889. A stepwise and practical approach for evaluating the structural integrity of historical and deteriorated steel bridges, incorporating analytical, mechanical and structural characterizations, is presented. Critical regions of hot-spot members were identified using structural finite element analysis, and fatigue reliability assessment analysis has been performed along with traffic estimation, taking into account various scenarios of traffic increase, in order to assess the possible remaining fatigue life. This analysis has evidenced that the structure might be kept in service at least for other 10 years. Appropriate retrofitting interventions are also indicated.

Keywords: railway; steel bridge; high-cycle fatigue; material tests; shear tests; rivets.

Introduction

A wide variety of steel bridges built before the 1930s made of pre-standard steel continued to serve the public railway Italian net. Historical iron bridges are considered to ex-

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pose users safety to risks, and for this reason they are often removed from service by railway authorities. According to a study conducted by the ASCE Committee on Fatigue and Fracture Reliability (1982), 80–90% of failures in steel structures are related to fatigue and fracture. A follow up paper of this study is reported in Byers et al. (1997).

At the same time, iron historical bridges represent a relevant category of the international cultural heritage, being the evidence of the modern industrial technology, particularly those intended to accommodate activities of an industrial or transport infrastructure. Many of these structures require particular rehabilitation due to design defects, basic elements deterioration, variation of use or change of the intensity of the imposed loads. With regard to Italy, the historical heritage is rich of significant metal structures, which played an essential role in the growth of industrial civilization: the most part of this heritage is represented by bridges, and the 60 per cent of Italian railway steel bridges has about one hundred years, as they were built between 1900th-1920th.

The first materials used were cast iron and wrought iron. The most common degradation phenomena in these bridges are in general related to the lack of proper maintenance, corrosion, structural details more sensitive to the phenomenon of fatigue than the actual ones, particularly in relation to the base materials used.

A common practice among railway authorities lays on the dismantling of historical bridges of about one century of service life, especially along principle line and if increasing loading demands and traffic volume has affected the bridge.

As a consequence, in order to find an alternative to the current practice that could lead to the dismantling of a relevant amount of in service historical bridges, accurate studies on remaining life prediction and fatigue assessment procedure are needed, because they may significantly change the use of a large amount of old steel bridges, maintaining them in service with no relevant interventions, and accurate monitoring procedure.

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The assessment procedure usually starts with in situ and laboratory tests: these allow to understand the material properties where they are not known as confirmed by recent studies (Ermopoulos 2006, Farhey et al. 1997): moreover, the precise steel material adopted could be investigated and, for e.g. specific interventions could be planned. Among all, some studies could be stated as reference in fatigue analysis, for e.g. Out et al. (1984) and Fisher et al. (1987), that reported the fatigue strength of deteriorated riveted built-up members of an 80-year old railroad bridge; Kulicki et al. (1990) studied the environmental effects of corrosion on fatigue strength of steel bridge beams. Moreover, the influence of specific factors in fatigue assessment has been documented by several researches, such as Bruhwiler et al. (1990), Kulak (1992), Akesson (1994), Di Battista et al. (1997), Matar and Greiner (2006), and Pipinato (2008). Concerning corrosion, for e.g. in Aktan et al. (1994), a riveted bridge of 1914 was investigated: field tests indicated that widespread corrosion with especially deep rust pits, distributed within some of the critical members and connections, did not affect the structural strength; conversely, the structural performance of the manufact was found to be affected by a brittle failure occurred at a location where it was not expected, due to an unanticipated mechanism.

In this paper the Paderno steel bridge, a typical arched railway of the mid-nineteenth century, is studied according to a step-level assessment procedure proposed in Pipinato (2008). First the bridge is geometrically described and a literature material investigation is carried out. Then, a linear FEM model is used to find out critical hot spot stress. Finally hot spot stress data are used in order to perform the reliability fatigue assessment and giving repair indications.

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The Paderno Bridge

Built between 1887 and 1889, designed by the Swiss engineer Jules Röthlisberger, then head of the *"Officine di Savigliano"* technical department, the Paderno bridge is very similar to the Gabarit bridge built in France four years earlier by Gustave Eiffel. Standing 85 m (238 feet) high, 266 m (745 feet) long and with a 150 m (420 feet) span, it could be defined a symbol of industrial archaeology in Italy. The bridge is currently used by the railway and the road traffic connecting Monza to Bergamo, as described in Figure 1.

Geometric and structural survey

The arch has a span of 150 m (420 feet), sag of 37.50 m (105 feet) and the principle girder is carried by means of 7 piers. The preliminary design of the Seregno-Ponte San Pietro railway line reported in Figure 2 represents the longitudinal profile and the construction site map of the bridge, while in Figure 3 the as built design of the front view and the plan of it are reported. The detail of one of the seven pier supporting the girder is reported in Figure 4. The girder is used both by railway and by railroad traffic: the first is carried in the superior part of the girder, while the second in the inferior one: accordingly to this description purpose, the girder structure front view and cross section is presented in Figure 5-7, while from the girder structure plan reported in Figures 8-9 is possible to understand the different way to carry those different traffic.

The net distance from the railway and the railroad is of 6.30 m. The main truss are spaced 5 m one to each other, and their height is of 6.25 m. The net railroad height is of 4.6 m, the deck is composed by floor beams spaced 3.325 m and two stringers carrying the rails, while the pavement is iron sheet made. The roadway is carried by the superior part of the main truss, is 5 m large, with two walk passages covered with stone panels 1 m large at both side; the way is transversally carried by the floor beams spaced 3.325 m and overhanging from the main truss, longitudinally by I beams 1 m spaced. Both the road and the railroad is provided by wind bracings down the relative deck, made by crossing L profiles visible for e.g. in Figure 8. The great arch is composed by two parabolic arches, the axis of which lay in a skewed plan, as could be observed in Figure 10: as a consequence, the truss arch is 5 m large at the mid-span and 16.346 m large at the bearings; the correspondent cross sections height are 4 m at the mid-span and 8 m at the bearings, and are made of riveted profiles. Chords are made of composed riveted couple of T sections, and are connected by a reticular system and by a couple of cross bracings. This arched structure lays on Moltrasio stone abutments, covered by Baveno granite and supported by cast iron, as reported in Figures 11 and 12. Every pier is composed by two skewed truss uprights, linked together by a rigid transverse and Saint Andrew cross frame. A foot walk 1 m large cross all the arch on the inferior part.

Degradation and retrofitting works performed

The first relevant damage of the bridge was due to bombing during the second world war. Damages interested in particular the third and the fourth span of the truss girder and one pier, but not the arch. All these damages were finally repaired in 1953, and the bridge was completely repainted in 1956 as widely documented in Bertolini (1989). In 1972 the roadway deck was substituted by an orthotropic deck, made by 15 mm plates and T reinforcing elements 30 cm spaced, as could be noticed in Figure 13. The last painting operation was developed 30 years ago, and the existing structure is not severely damaged by corrosion. Maintenance works have been extended to the whole bridge but have not been systematic. Some chords portions are difficult to reach and consequently have not been re-painted recently: these appear black and are still in their original condition. The U-shaped arch section of the lower chords and the related joints are open up-

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wards so that they carry large quantities of water during raining, and the drain system is inadequate. Anyway, corrosion has not deeply attached the principle members, even if paint raisings are diffused and superficial corrosion is extended. Also local deformation in thin plates and rivets popping off have been observed diffusely on the structure. The absence of regular spaced thermal expansion joints onto the roadway has led to cracking on the pavement. Actually, due to the absence of a deep knowledge of the real structural situation, the railway authority has imposed a load limitation of 16t and also speed has been limited. In Figure 14 the front size of the bridge is reported.

Structural analysis

Material characterization tests

Old metals do not usually fulfill the precise requirements of normalized materials according to EN 10025 (2004). As a result the knowledge of the material properties of existing metal bridges is essential for the resistance assessment and the determination of the remaining lifetime of the bridge (ORE 1986; Liechti et al. 1997; ICOM 2001; ECCS 2008; Sustainable Bridges 2006). Moreover, for old metal bridges that were built between 1870 and 1940 in particular, the material parameters are in many cases not available. Sustainable bridges (2006) suggests that the highest attention to the mechanical properties must be paid when dealing with wrought (puddle) iron and old steels, because due to the production process these irons have a large amount of slags and inclusions, plus a great anisotropy. According to these preliminary suggestions, even if in situ or NDT testing have not been performed, literature testing results have been reported and are discussed in the following. Metallographic test (Bertolini 1989) evidenced a ferritic structure with non-metal inclusions and with a fibrous structure. Chemical analysis performed by Bertolini (1989) are reported in Table 1: this has evidenced a

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very low carbon content confirming the ferritic structure, and an high content of phosphorous if compared to the actual coding EN 10025 (2004). All these evidences, lead Bertolini (1989) to confirm that the material is a puddle iron. Concerning to this particular material, a particular attention has been suggested by Sustainable Bridges (2006) as the structural behavior against fatigue of these type of metal is very brittle and could lead to sudden cracking. Moreover, old riveted steel members could be classified in the fatigue detail category C=63 (ECCS 2008; Sustainable Bridges 2006). Concerning the tensile and Charpy toughness test, those are reported in Tables 2-3 and were made in 1955 and 1972 (Bertolini 1989): results are similar to a S235 steel concerning yielding and toughness (EN 1993-1-1, 2005), even if the tensile strength is lower. Other tensile tests are available in the same study, confirming for all the structural details investigated, both the arch and the girder, the same results. No testing on rivets have been found.

Structural model

A detailed FEM model of the entire bridge has been performed, assuming the nomenclature reference reported in Fig. 15 and 16, as could be observed in Figure 17. No alternative model has been realized computing for material degradation (e.g. reducing transversal section), as the visual inspection performed has considered this problem not affecting deeply the structure. As the aim of the bridge assessment is referred to the in service conditions, all the models are assumed elastic. Some characteristics related to the Fem model realized are: (i) it has been decided not to take into account the relative positions of different beams converging into joint node, because some sub-models realized has confirmed that this has no influence on the flexural stiffness; moreover, the use in static models of a medium baricentric line is admitted by the Instruction 2298 (1997)

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of the national authority; (ii) truss element have been used for structural elements, except for gusset and joint plates in which plates elements have been adopted; (iii) all section geometry for each beam element have been shaped as the as built section, assuming no bending transmission from one to each other; (iv) transversal actions, as wind and hunting, is assumed to be carried by the transversal inferior and superior bracings of the girder; (v) stringers and floor beams have been assumed to be simply supported. The model has been calibrated with the deformation obtained by the in situ definitive test performed in 1892, with a distributed load of 5.1 t/m realized with six locomotives and tenders (450 bis type), for a total load of 500.22 t: the comparison between these values have been reported in Table 4.

Fatigue reliability assessment

Assuming the dead load as a reference state, the effect of load cycles is represented by the fluctuation $\Delta\sigma$ of the stresses induced by the passing trains. Secondary stress fluctuations are far below the limit for cumulative damage, i.e. allowing a single value for the stress fluctuation to be taken into account.

As reported in Sustainable bridges (2006), assessment could refer to the whole infrastructural line, to the single bridge, or to a specific detail. The step by step evaluation adopted in the research mentioned above (Pipinato 2008) and here partially reported, could be summed up by the following stages:

- study and inspection of design documents and check of their correctness;
- preliminary inspection in order to identify the structural system and possible damages;
- supplementary investigations in order to refine information about the bridge;

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- structural assessment in order to evaluate load carrying capacity and safety of the bridge.

Concerning this last topic, the fatigue step level assessment procedure is articulated in:

a) deterministic assessment;

b) simplified probabilistic assessment;

c) full probabilistic assessment.

According to this scheme, the fatigue assessment of the bridge analyzed in this paper have been developed. In the following the fatigue model discussion, the load model adopted, cycle counting considerations, and finally the three step of the assessment are presented.

Fatigue model

In order to assess bending details for riveted connections, specific detail categories have been adopted: for bending detail, category C=63 has been assumed, with reference to Sustainable bridges (2006) findings; for shear detail, category C=100 has been assumed as suggested by EN 1993-1-9 (2005).

Load model

The fatigue deterministic damage assessment was performed by using a semi-probabilistic approach through both the equivalent stress method and the cumulative damage method (Miner, 1945) as suggested by EN 1993-1-9 (2005). Moreover, in order to provide further estimates, the recent Sustainable bridges (2006) guidelines was employed. According to Italian National codes, fatigue analysis has to be performed over 10.000 load cycles, an amount greatly exceeded by the number of cycles performed. Fatigue assessment has to rely on approximate data concerning historical traffic, since no

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direct survey on the bridge has ever been performed. The bridge has been re-opened to traffic in 1946 and has been in service up to now, so for the historical traffic has to be taken into account only the last period of 64 years. The fatigue load used to perform the fatigue verification has been adopted with the following parameters:

- EN 1993-1-9 (2005) load model: this load spectrum has been considered to be effective since 1990; historic loads has been considered as provided by the Authority, so from 1945 till 1990, 13.2 t/m locomotives and 8 t/m carriages has been considered;
- Sustainable bridges (2006) consider a particular distribution of growing loads, that is reported in the following.

According to EN 1993-1-9 (2005), the fatigue check with the equivalent stress method entails the use of the following ratio,

$$\eta_{EC_a} = \frac{y_{F_f}\,\Delta\sigma_{E_2}}{\dfrac{\Delta\sigma_C}{y_{M_f}}} \leq 1.0, \quad (1)$$

where η_{EC_a} – fatigue ratio for EN 1993-1-9 verification procedure; y_{F_f} – partial factor for equivalent constant amplitude stress ranges $\Delta\sigma_{E_2}$, [MPa]; $\Delta\sigma_{E_2}$ – equivalent constant amplitude stress range related to 2 mln. cycles [MPa]; $\Delta\sigma_C$ – reference value of the fatigue strength at N_C = 2 mln. cycles, [MPa]; y_{M_f} – partial factor for fatigue strength $\Delta\sigma_C$.

Conversely, the cumulative damage approach implies the use of the formula

$$D_{d,EC_a} = \sum_i^n \frac{n_{E_i}}{N_{R_i}} \leq 1.0, \quad (2)$$

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where n_{E_i} – the number of cycles associated with the stress range $y_{F_f}\Delta\sigma_i$ for band "i" in the factored spectrum, [MPa]; N_{R_i} – the endurance (in cycles) obtained from the factored $\dfrac{\Delta\sigma_C}{y_{M_f}}$ vs. N_R curve for a stress range of $y_{F_f}\Delta\sigma_i$, [MPa].

The corresponding rail traffic action adopted in these formulae was the load model LM 71 defined in EN 1991-2:2005 "Eurocode 1: Actions on Structures. Part 2: Traffic Loads on Bridges".

With regard to the Sustainable Bridges:2006, the fatigue safety of all fatigue vulnerable details must be based on fatigue safety ratios. In detail, two ratios were considered:

- the fatigue limit $\eta_{ED4.2}$, i.e. the limit below which no crack propagation occurs, that reads,

$$\eta_{ED4.2} = \frac{\dfrac{\Delta\sigma_D}{y_{f_{at}}}}{\Delta\sigma_{\max}} \geq 1.0, \tag{3}$$

where $\eta_{ED4.2}$ – fatigue ratio for Sustainable Bridges:2006 verification procedure; $\Delta\sigma_D$ – fatigue limit of the investigated construction detail, [MPa]; $y_{f_{at}}$ – fatigue resistance coefficient; $\Delta\sigma_{max}$ – max fatigue action effect (stress range) [MPa];

- the fatigue strength $D_{ED4.2}$, for which crack propagation occurs, provided that $D_{ED4.2}$ turns to be smaller than one,

$$D_{ED4.2} = \frac{\dfrac{\Delta\sigma_C}{y_{f_{at}}}}{\Delta\sigma_e} \geq 1.0, \tag{4}$$

where $\Delta\sigma_C$ – fatigue strength at 2 mln. cycles (fatigue category), [MPa]; $\Delta\sigma_e$ – (equivalent) fatigue load effect referred to 2 mln. of cycles, [MPa].

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In this case, the relevant Rail traffic actions considered and the load levels suggested are those of the Sustainable Bridges (2006) guidelines, as depicted in Fig. 18 and Table 5.

Cycle counting

In order to perform a detailed damage assessment, an estimation of cycles affecting each single detail was carried out: this estimate was performed by making tension/time fluctuations analyses for each detail and loading spectrum, as well as by counting the effective cycles as per ASTM E1049-85 (2005).

Deterministic assessment

Results of the deterministic assessment is reported in Table 6 for the hot spot details observed during the analysis (see Fig. 19). While details 501, 702 and 901 are in correspondence to the arch constraint, details 4 and 5 are substructure of all the spans of the bridge girder pertaining to the railway line; in particular, detail 4 is the riveted connection of the floor beam to the lower chord and detail 5 is the lower chord at mid-span.

As evidenced from this analysis, the fatigue life of riveted railway bridges is governed by particular critical structural details that undergo a much larger number of loading fluctuations and of stress variations upper the cut-off limit with respect to other members. In this case, they are represented by riveted connections.

While the assumption made by Sustainable bridges (2006) appears to reflect the actual evolution of railway loadings, on the other hand, the constant load levels of 225 kN suggested by the load model LM71 of EN 1991-2 (2005), considered for the whole life of the bridge, led to an overestimate of the fatigue life for all details.

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Simplified probabilistic assessment

The probability of crack detection during inspection and monitoring is generally evaluated in an intermediate stage, and subsequently linked to the (calculated) probability of fatigue fracture to obtain the probability of failure according to Sustainable Bridges (2006):

$$p_{fail} = p_{fat}\,(1 - p_{det}) \tag{5}$$

where p_{fail} – probability of failure; p_{fat} – probability of fatigue fracture; p_{det} – probability of detection.

The probability of failure can also be expressed by means of the reliability index according to the standard normal distribution (Benjamin, Cornell 1970). Finally the reliability of a structural element is compared to the target value:

$$\beta_{fail} \geq \beta_{target}$$

where β_{fail} – reliability index with respect to failure; β_{target} – target reliability index.

This model adopt the fatigue action effect (the required nominal fatigue strength) as "required operational load factor α_{req}" which is obtained by dividing the required nominal fatigue strength by the action effect of the fatigue load, consisting of the load model UIC 71 (Kunz 1992):

$$\alpha_{req} = \frac{\Delta\sigma_{C,req}}{\Delta\sigma(\Phi Q_{fat})}, \tag{6}$$

where $\Delta\sigma_{C,req}$ – required nominal fatigue strength, [MPa]; α_{req} – required operational load factor; $\Delta\sigma(\Phi Q_{fat})$ – stress range due to load model UIC 71 at worst position [MPa].

For a simplified probabilistic approach, it is here adopted a relation between mean value of required operational load factor $m(\log\alpha_{req})$ and number of future train passages N_{fut} – established by Kunz (1992) using the action effect of the traffic model in UIC 779-1 (1986) including assumptions on the scatter. The 1990 year was taken as the reference

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time from which all future trains are counted ($N_{fut} \geq 1$). The mean of required operational load factor $m(\log\alpha_{req})$ is then read for three different ranges of fatigue categories (expressed as N_D, the cycle number of fatigue limit) starting from a defined number of future trains N_{fut} (as from 1990). This procedure is suitable for influence lengths over 10 m, a commissioning time 1900 ± 25 and a partition of freight traffic of 75%. There is one relation for 60 tpd (trains per day) and one for 120 tpd in the past (before 1990). According to the same model, a value of 0.04 may be taken as standard deviation of the required operational load factors, resulting from the assumed fuzziness of the traffic model. Adopting the following notation and assumption:

$$s_E = s(\log\alpha_{req}) = 0.04, \quad (7)$$

$$\beta_{fat}(N_{fut}) = \frac{m_r + m_E(N_{fut})}{s_r^2 + s_E^2} \quad (8)$$

where s_E – the standard deviation of the required fatigue strength; $\beta_{fat}(N_{fut})$ – reliability index; $m_R = \log\Delta\sigma_c + 2s_R$ – the mean of the fatigue strength ($\log\Delta\sigma$ relating to $N = 2 \times 10^6$, [MPa]); $m_E(N_{fut}) = m(\log\alpha_{req}) + \log\Delta\sigma(\Phi Q_{fat})$ – the mean of the required fatigue strength as a function of the number of future trains N_{fut}, [number of cycles]; $s_R = \frac{s(\log N)}{m}$ – the standard deviation of the fatigue strength, [MPa]; m – the slope of the S-N curve; s(logN) – the standard deviation of test results.

Concerning the choice on the target reliability index, it has been considered that for Serviceability Limit States specific values of β are recommended for a determinate remaining service life, according to ISO (1999): for the fatigue limit state and a remaining service life of 50 years, a value of β = 2.3 is recommended in case of inspection and β = 3.1 – if the element or detail is not inspectionable (ISO 1999). Therefore, indexes adopted here are the following: $\beta_{max} = 3.1$ – detail inspectable and $\beta_{min} = 2.3$ – detail not inspectable.

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Graphs reported referred to an analysis dealing with a medium annual passage of 50.000 trains (overestimating the period 1990–2009), a number of 60 tpd (train per day) before 1990, and with the other fixed following parameters: slope S-N curve m = 3; standard deviation of test result, s(logN) = 0.45 (Kunz 1992); standard deviation of the required fatigue strength, s_E = 0.04. According to the aforementioned fatigue model, a possible new category C = 110 for shear has been introduced, as suggested by recent findings (Pipinato et al. 2009). Results are reported in Fig. 20 and in Fig. 21 with reference to detail category of EN 1993-1-9, and in Fig. 22 adopting the suggested detail category C=110 (15.95 ksi circa), for a max stress calculated excursion $\Delta\sigma \times \phi$ = 91 MPa (13.2 ksi circa) for the inferior flange at the midspan (Detail 5) and $\Delta\tau \times \phi$ = 132 MPa (19.15 ksi circa) for the shear riveted connection of the floor beam (Detail 4).

Results highlights a similar scenario of the previous observation made on deterministic assessment: detail 4 results to be the more dangerous detail against fatigue. These results have been obtained, according to the experienced life from 1946 until today and of an average number of 50.000 tpd (trains per year) from 1990 to 2009 (137 tpd). The remaining life of the entire bridge has then been calculated for the hot spot detail assuming different increasing traffic for the future and category detail C=100 (14.5 ksi circa): an estimation of the remaining life of the bridge, focusing on three trends type of future traffic demands, with an increment of the 5%, 10% and 15% of traffic from 2009 every 5 year is reported in Table 7; no increasing in loads has been taken into account, and the remaining life has been calculated basing on the hot spot detail 4: when the reliability index goes down the lower limit β_{min} =2.3, the bridge is considered out of service.

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Full Probabilistic Assessment

Structural reliability analysis, as an evaluation of the failure probability usually deals with a not correlated variables, all describing the ageing situation of a structure in general, of a bridge in our case. The use of more than two variables imply the use of a transformed function: this requires the introduction of a series of simplifications in order to avoid direct multidimensional integration, or the use of simulations techniques. A simple quantification of the structural reliability can be done using the same concept of reliability index seen before, applied to multiple variables. As the reliability of a bridge may involve the presence of multiple variables such as dead load, live load, compressive strength, yield strength, geometric dimensions, crack growth etc, the limit state function can be expressed as a function of these variables. The definition of the reliability index in the two-variables case can be generalized also for n basic variables. This method is known as MFOSM, or Mean-value First-Order Second-Moment method: the Taylor expansion is made about the mean values point and is truncated at the first order terms, and only the first two moments of the distributions are required. The method has an invariance problem: the value of β depends on the shape of the limit state function. The so-called FOSM method addresses this problem by making the Taylor expansion about an unknown point of the limit state function (design point), found by means of an iterative process. MFOSM and FOSM methods do not require any knowledge on the random variables distribution type, but because of this the information on reliability associated with a given value of β is limited and the probability of failure remains undetermined. Otherwise, possible applicative methods for full probabilistic evaluation specifically for metal bridges could be found in Kunz (1992) or in Boulent et al. (2008), and are more theorically deepen in Frangopol and Moses (1994), Frangopol and Maute (2003) and Petcherdchoo et al. (2008). The disavantages of the reliability analisys are

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the increased complexity of calculations, the large amount of input data needed (which may be or not available) and the ability required to influence the results by manipulating the input data as confirmed by Estes and Frangopol (2005).

Retrofitting works

As a result of the aforementioned assessment, due to the high degree of redundancy discovered both in the structure as a whole and in the joints, the Paderno bridge steel structure is not in a dangerous situation, nor it would be in the next years. Anyway, some non redundant hot spot fatigue detail has been discovered and should be carefully monitored. The amount of localized severe corrosion, and extended superficial damage lead to the conclusion that superficial treatment and painting of principle beams would be necessary in order to prevent higher section loss. Other retrofitting works, that would be possible only throughout an on site visual assessment of all elements, will be related to:

- accurate repainting of internal part of lower chords;
- replacing of the most corroded elements as hangers and bracings;
- local repairing of floor beams throughout low-flange coverplating;
- replacing riveted connection loss with high strength bolts and coverplating works on connecting plate.

Conclusions

In this paper the assessment of the Paderno bridge has been presented by means of a stepwise procedure: (i) mechanical characterization of the materials; (ii) survey of the damaging effects; (iii) setting up of mechanical models, (iv) fatigue assessment , (v) definition of retrofitting works. The application of such a procedure to the bridge outlined some relevant issues: the materials used in this old railway bridge have shown mechanical and chemical properties (strength, chemical composition) not in line with those provided by contemporary codes; the high degree of redundancy of riveted railway girders makes the internal forces and stresses in the structural elements generally lower than in contemporary built structure; some non redundant hot spot fatigue detail has been discovered and should be carefully monitored; corrosion is a local phenomenon that does not necessarily play a relevant role on the global response of the bridge; fatigue cracks, not discovered in this first phase assessment, have to be taken into account in a deeper assessment phase, monitoring carefully structural details as floor hangers, stringer-to-floor beam connections, short diaphragms and riveted connections; fatigue assessment concerning the remaining life could be performed by using the code suggestion, but taking into account that historical traffic has to be modelled as an increasing quantity and not as a fixed value: results have given a reasonable and detailed estimation of safe exercise, using for e.g. Sustainable Bridges (2006) load model suggestions; moreover, simplified probabilistic assessment could suggest the remaining life of a bridge, accounting also the estimation of the residual lifetime with different scenario of traffic increase. The application of such a comprehensive procedure to the Paderno Bridge showed that a detailed analysis and assessment phase succeeded in limiting the retrofitting works to a small fraction of the costs needed for replacing the entire bridge. Therefore, detailed structural fatigue assessment should be considered economically convenient and would have to be adopted by a wide amount of infrastructure authorities.

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Table 1: Quantometric chemical test results.

Component	Average [weight %]
C	0.01
Mn	0.13
Si	0.07
S	0.032
P	0.41

Average values of 3 tests. Fe % is the remaining part.

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Table 2: Tensile test results.

Paderno Bridge (1955)	Yield strength [MPa]	Tensile strength [MPa]	Elongation A [%]
1	236	355	27
2	255	366	17
3	257	362	18
4	260	378	21
5	249	349	14
6	254	283	6
7	246	345	14
8	287	313	4
9	278	360	7
Medium value	258	346	14
Paderno Bridge (1972)	Yield strength [MPa]	Tensile strength [MPa]	Elongation A [%]
1	228	296	8
2	229	262	4
3	336	347	27
4	229	349	23
5	227	244	2
6	240	254	13
Medium value	248	292	13

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Table 3: Toughness test results.

Paderno Bridge (1972)	Section geometry [mm]	Temperature [°C]	Average impact energy [J]
C_1	10X10	20	12
C_2	10X10	‘’	44
C_3	10X10	‘’	34
C_4	10X10	‘’	29
C_5	10X10	‘’	40
C_6	10X10	‘’	28
C_7	10X10	‘’	32
Average value			31

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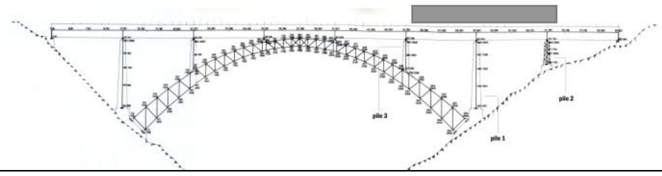
Tab. 4: Comparison between in situ test (1892) and results of the FEM model.



Test id (loaded span)		Measured displacement [mm]	FEM model displacement [mm]
1 (Span 2, 3)	Column I	-8.6	-9.5
1 (Span 2, 3)	Column II	-4.0	-5
1 (Span 2, 3)	Vertex	-	-
1 (Span 2, 3)	Column III	+3.6	+3.1
1 (Span 2, 3)	Column IV	+2.6	+2.2

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Table 5 Equivalent Freight Train according Sustainable Bridges:2006 : axle loads and numbers of axles per wagon.

Year	<1920	1921-1940	1941-1960	1961-1980	>1980	Mean
Speed	50	70	80	100	120	100
P_k	160	180	200	200	225	200
P_m	120	150	160	160	180	160
A	2	3	4	4	4	4
P_0	40	40	50	50	50	50

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Table 6. Deterministic assessment results.

Structural component	Detail id.	EC3 procedure		Sustainable bridges procedure	
		η_{EC3}	$D_{d,\ EC3}$	$\eta_{ED4.2}$	$D_{ED4.2}$
Arch	501	0.62	0.7	1.4	2
Arch	702	0.54	0.6	1.6	2.1
Arch	901	0.56	0.8	1.5	2.05
Girder	4	0.75	3.1	1.1	1.7
Girder	5	0.65	0.7	1	1.5

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Table 7. Traffic increase vs. remaining life.

Traffic increase	Remaining life
5%	19
10%	15
15%	10.5

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Fig. 17 Structural model of the bridge.

Fig. 18 Equivalent Freight Train according Sustainable Bridges:2006 : axle spacing and length.

Fig. 19 : Hot spot details of the investigated bridge.

Fig. 20. Probabilistic modelling of the likelihood of failure, detail 5 (bending detail category with reference to EN 1993-1-9 (2005), C=63).

Fig. 21. Probabilistic modelling of the likelihood of failure, detail 4 (shear detail category according to EN 1993-1-9 (2005), C=100).

Fig. 22. Probabilistic modelling of the likelihood of failure, detail 4 (experimental shear detail category C=110).

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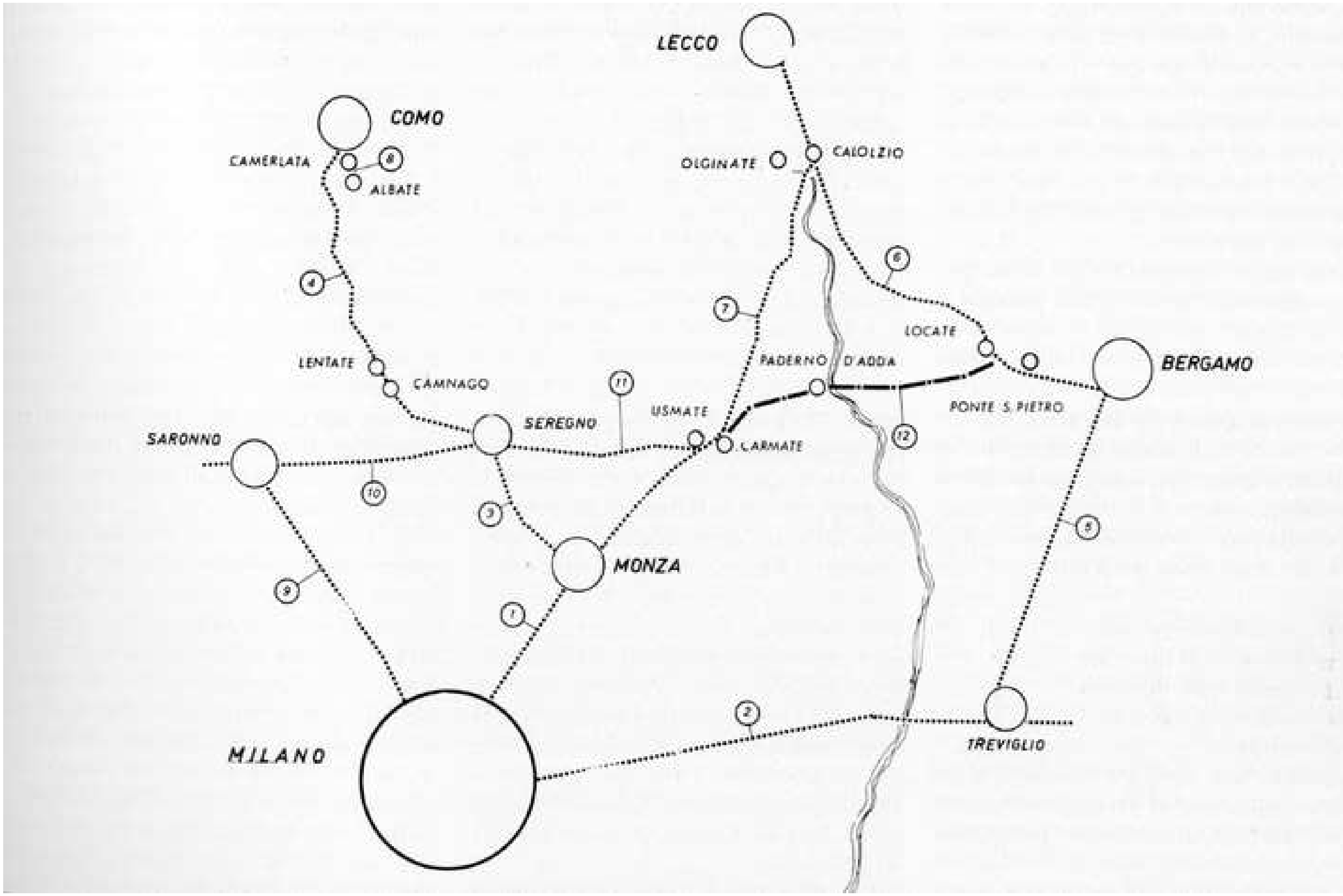

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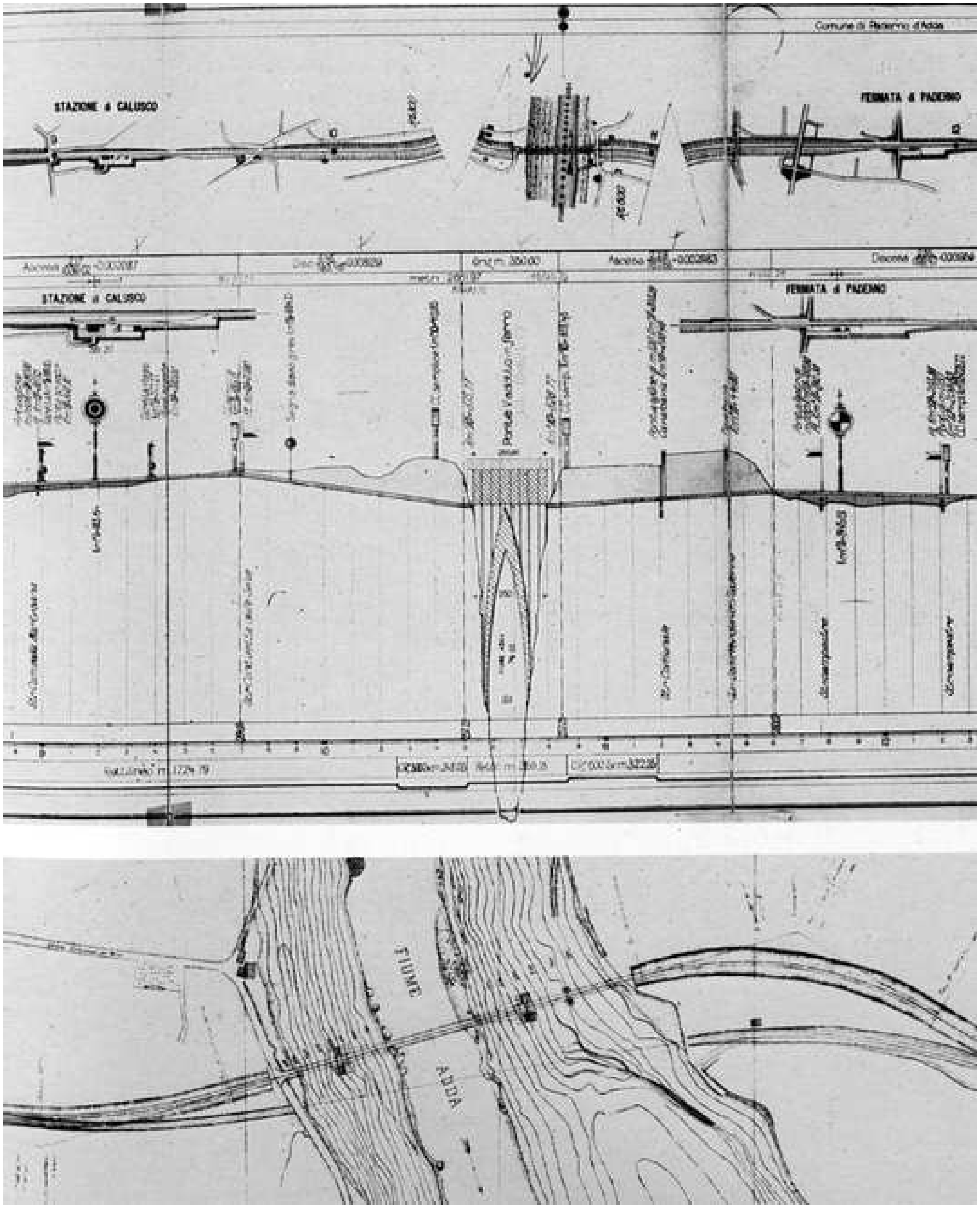

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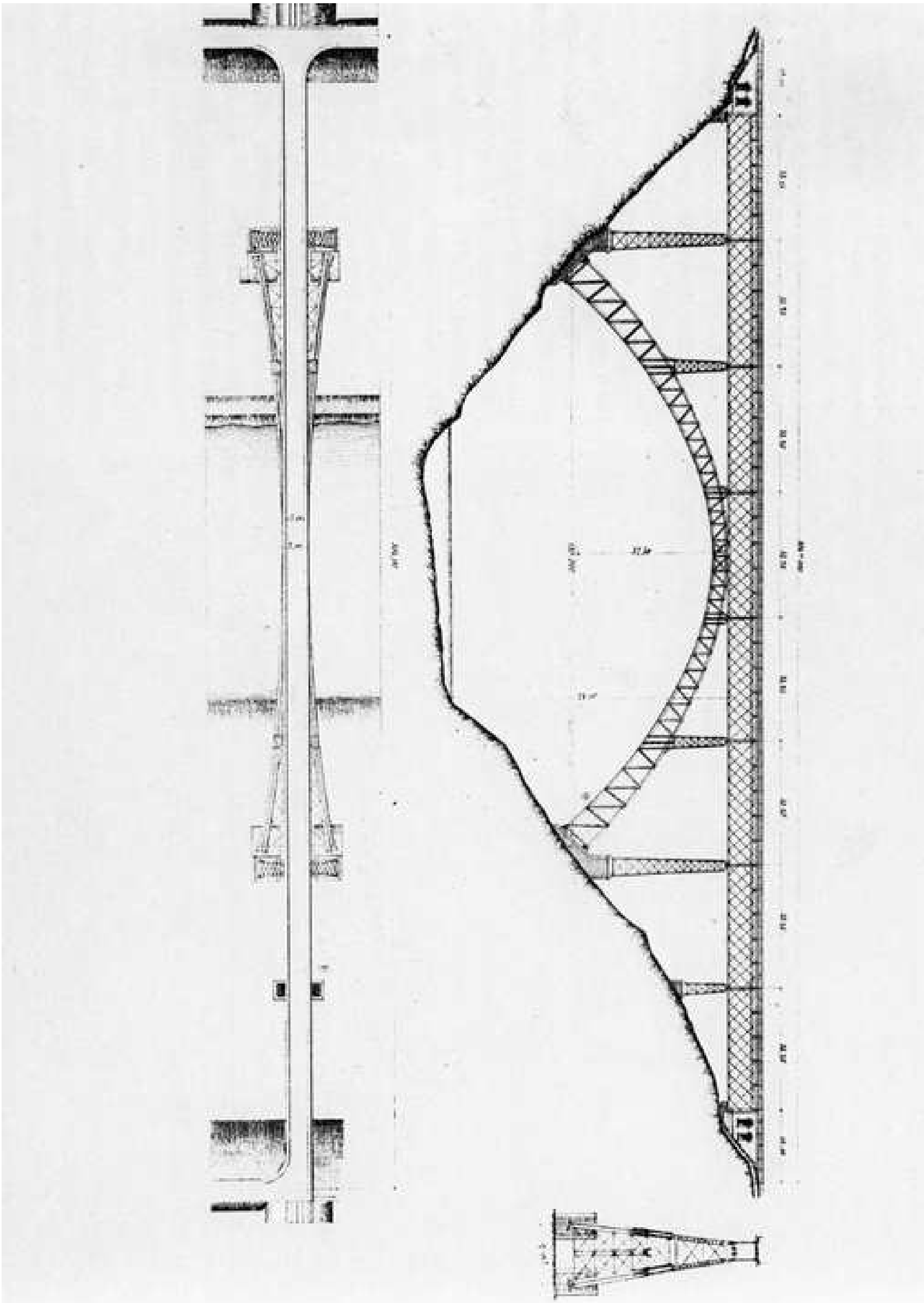

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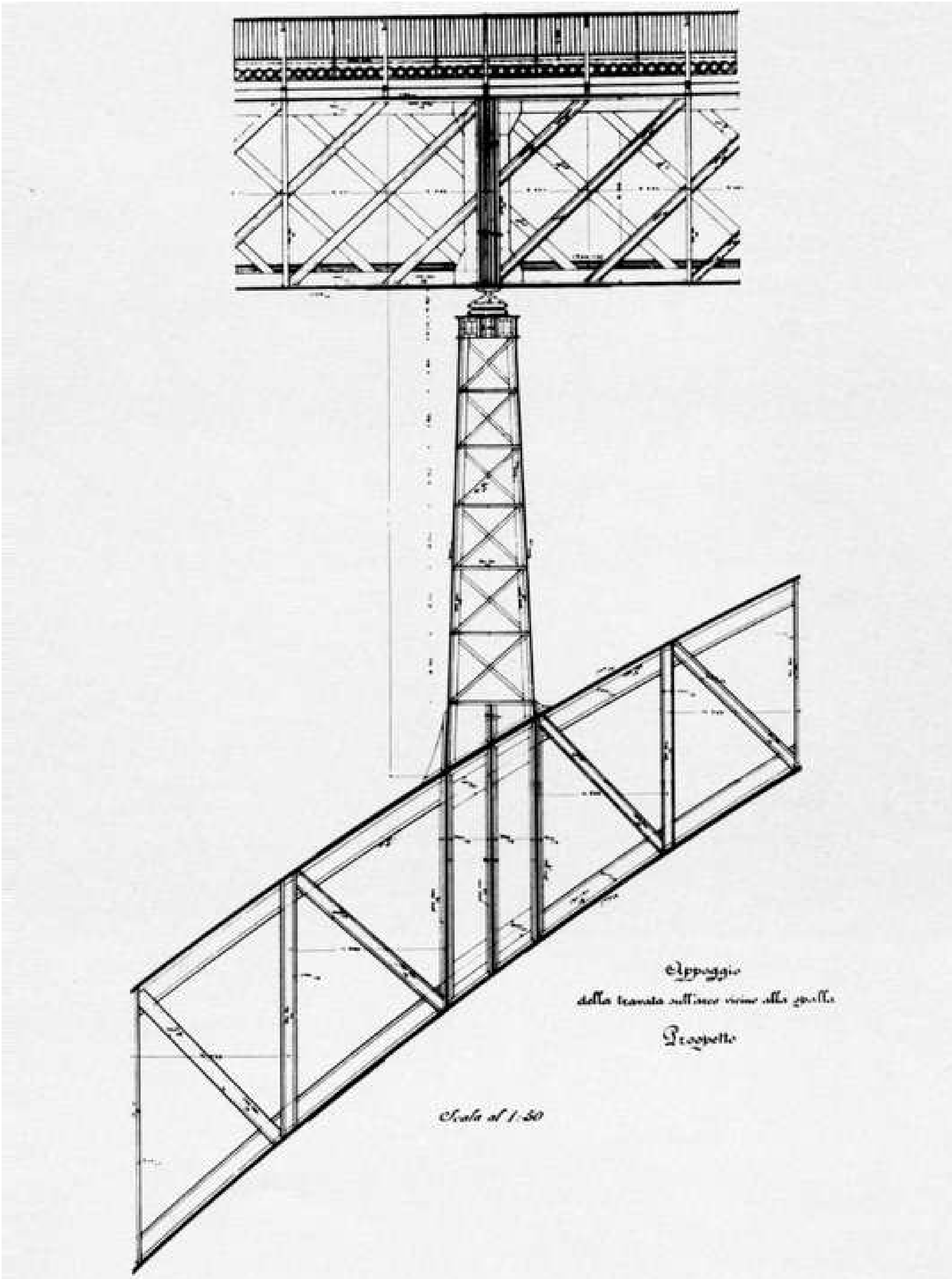

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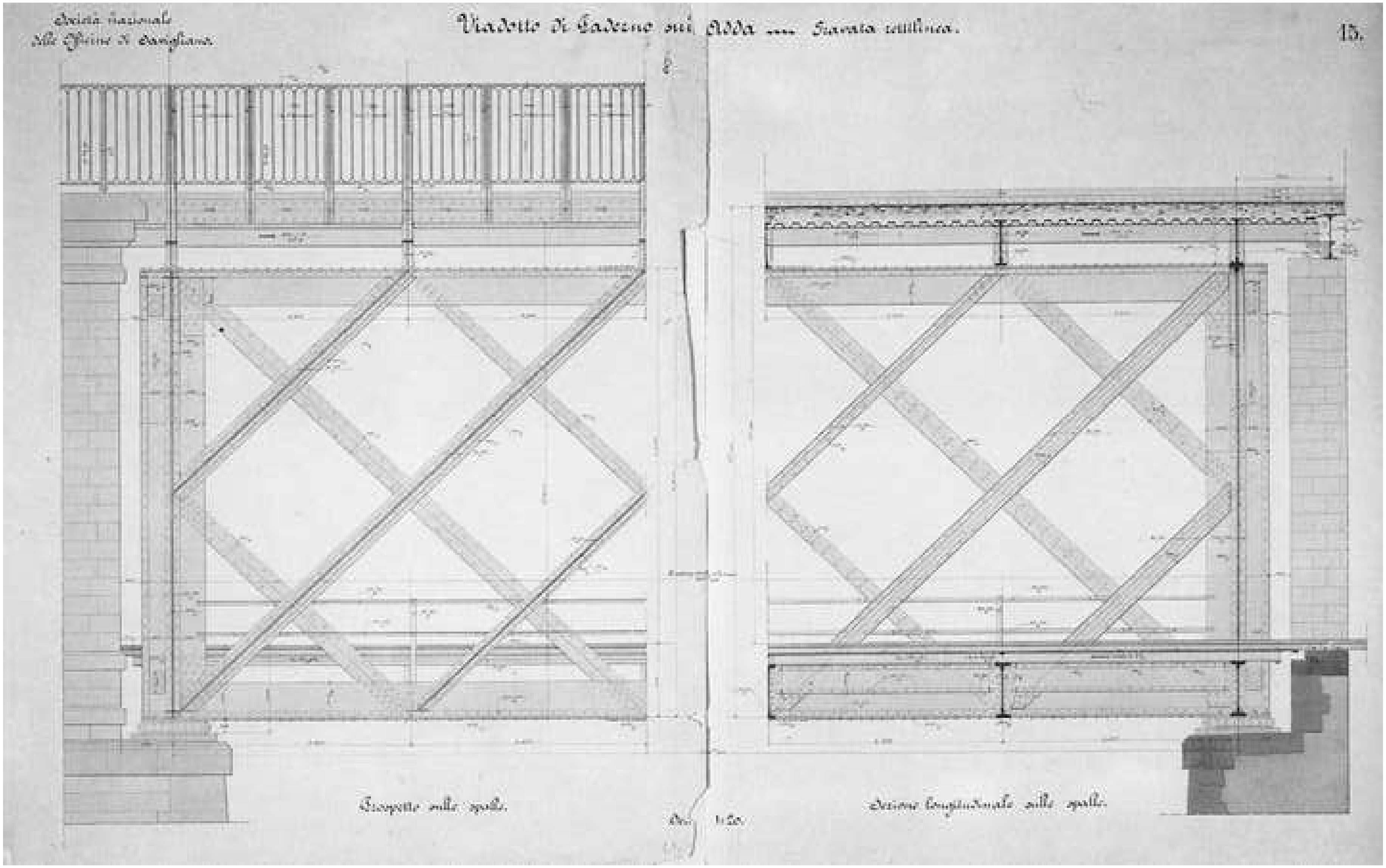

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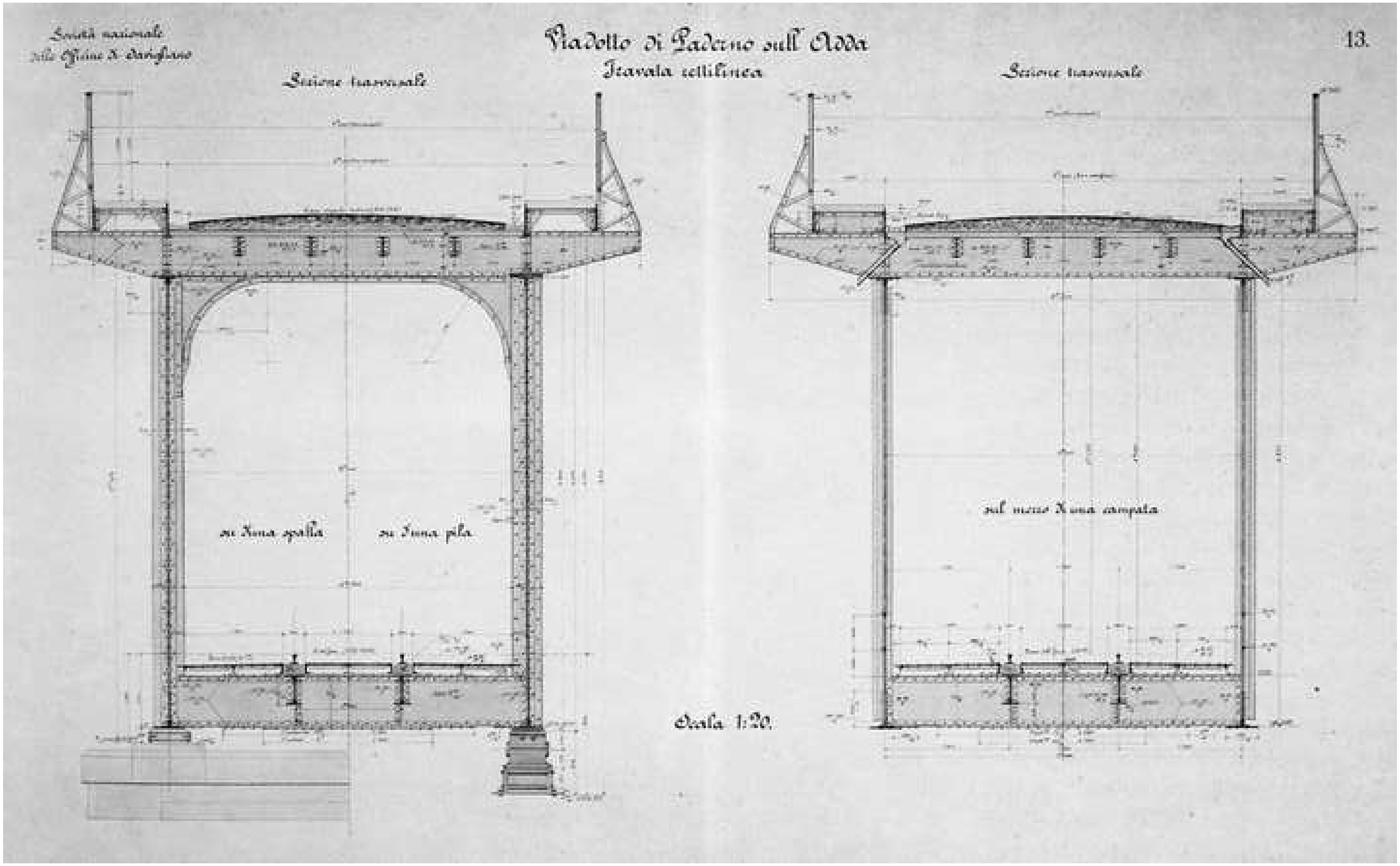

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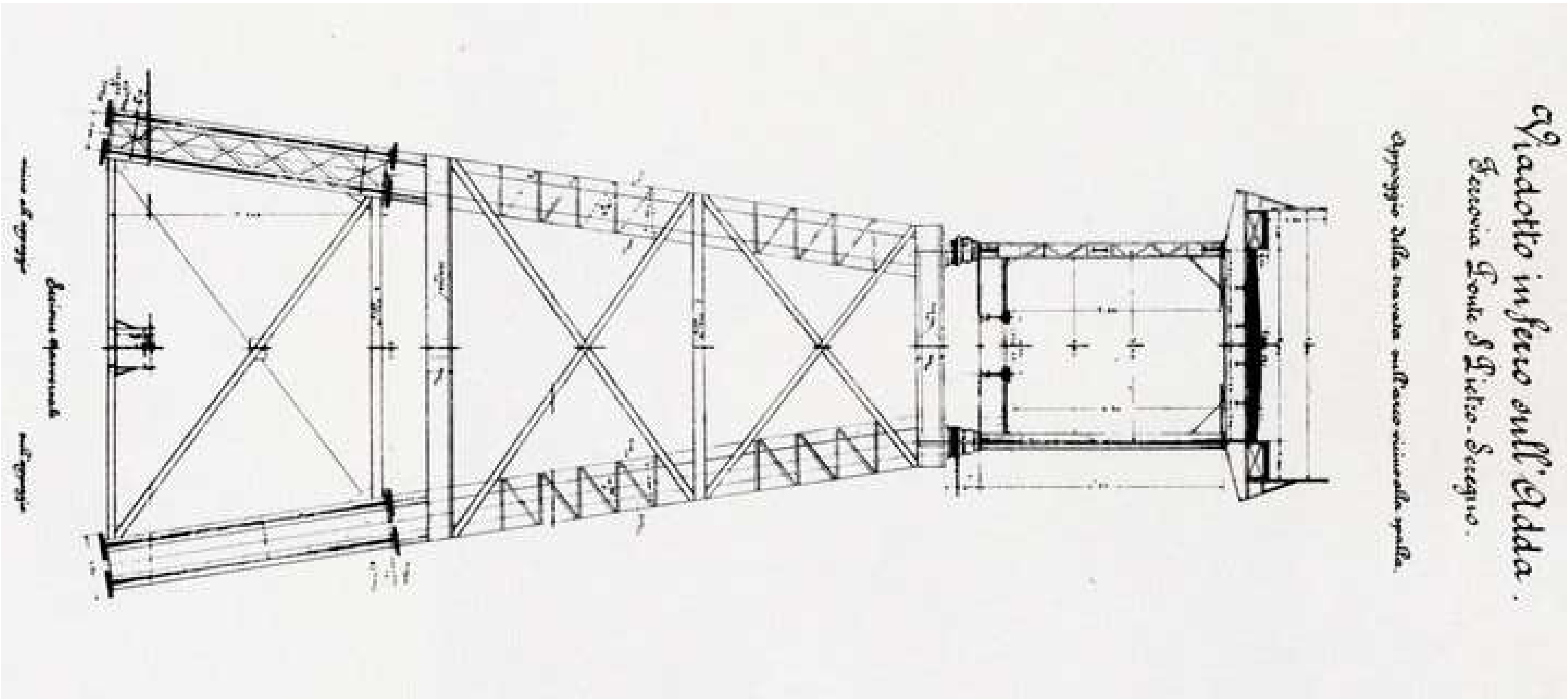

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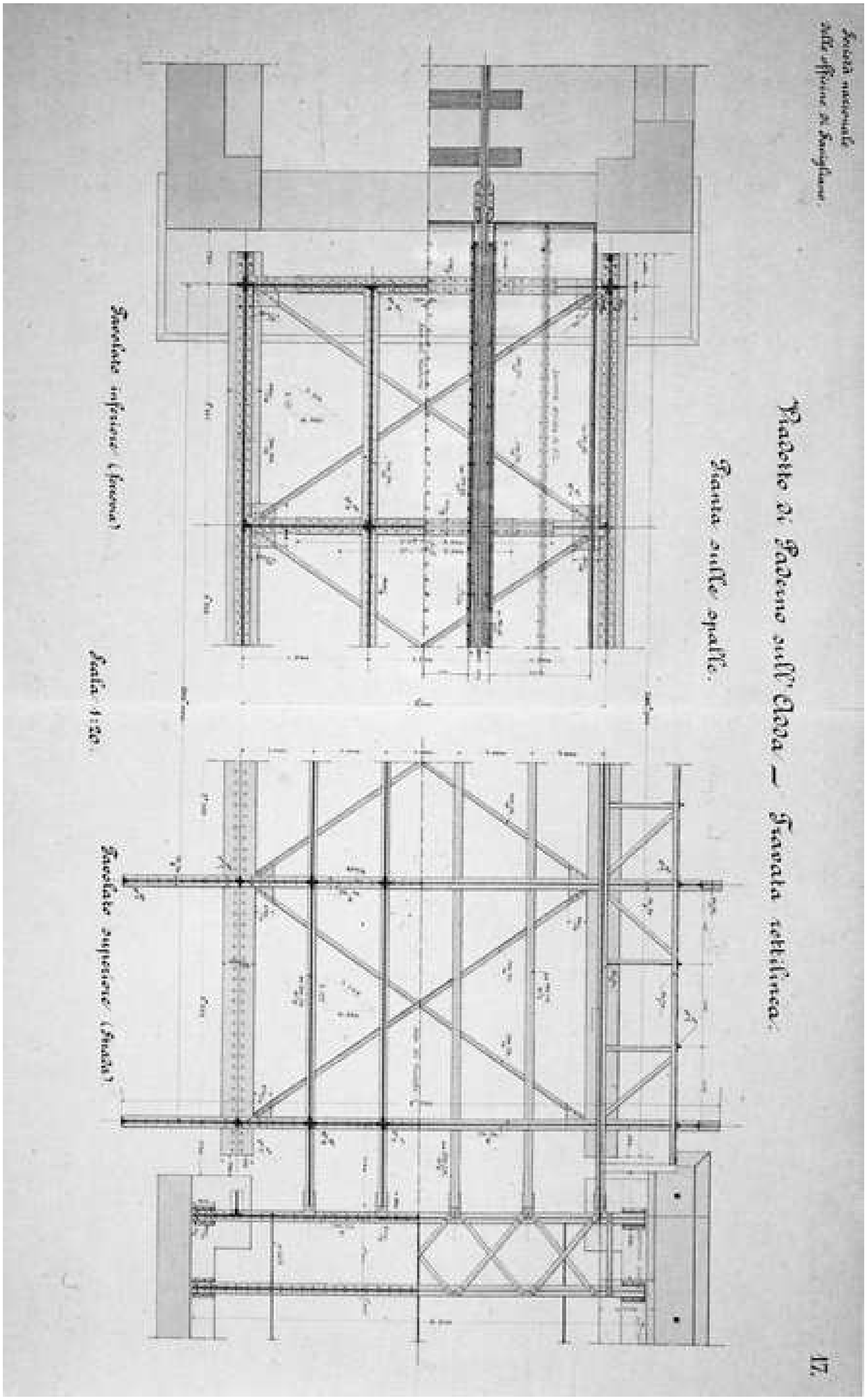

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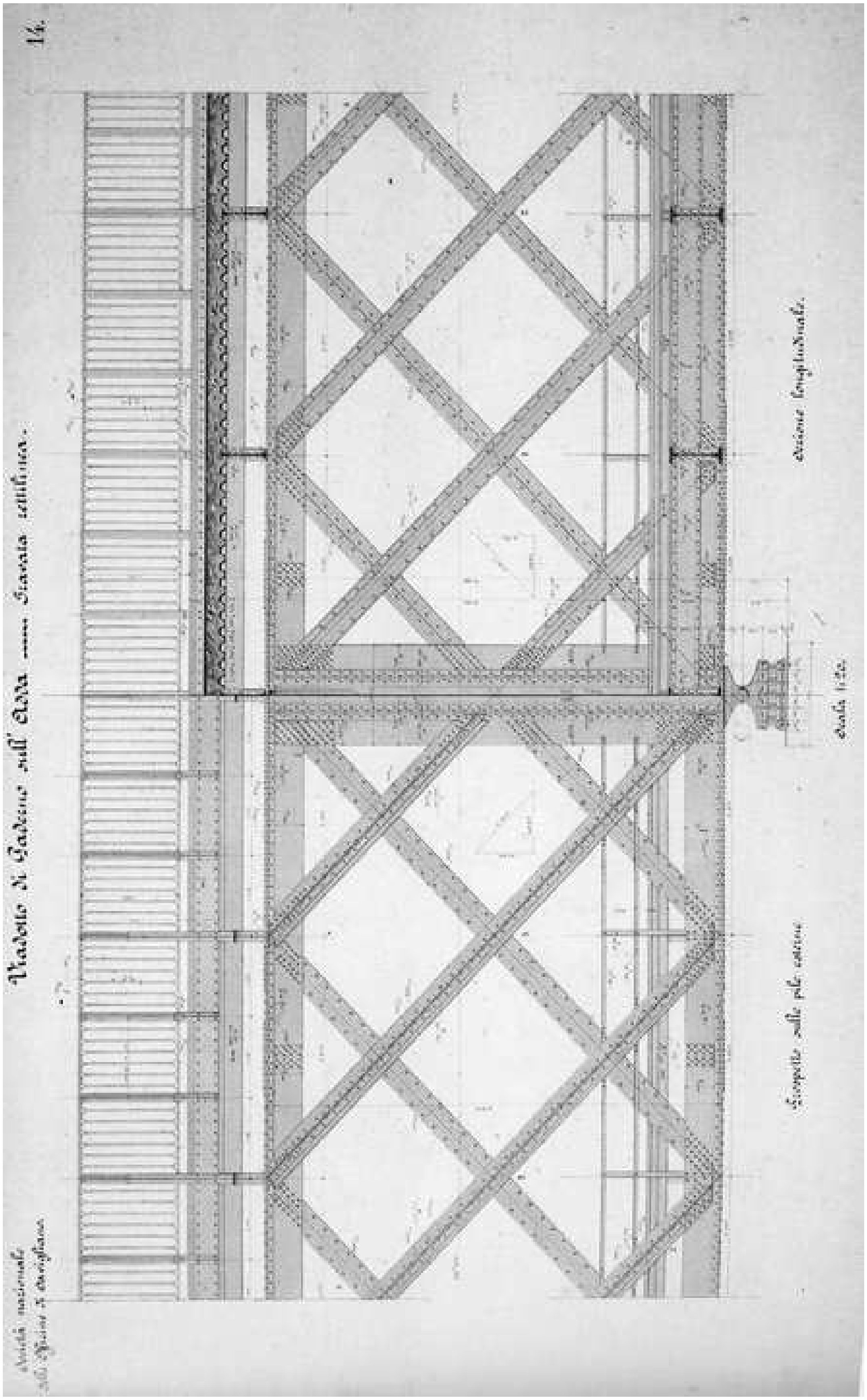

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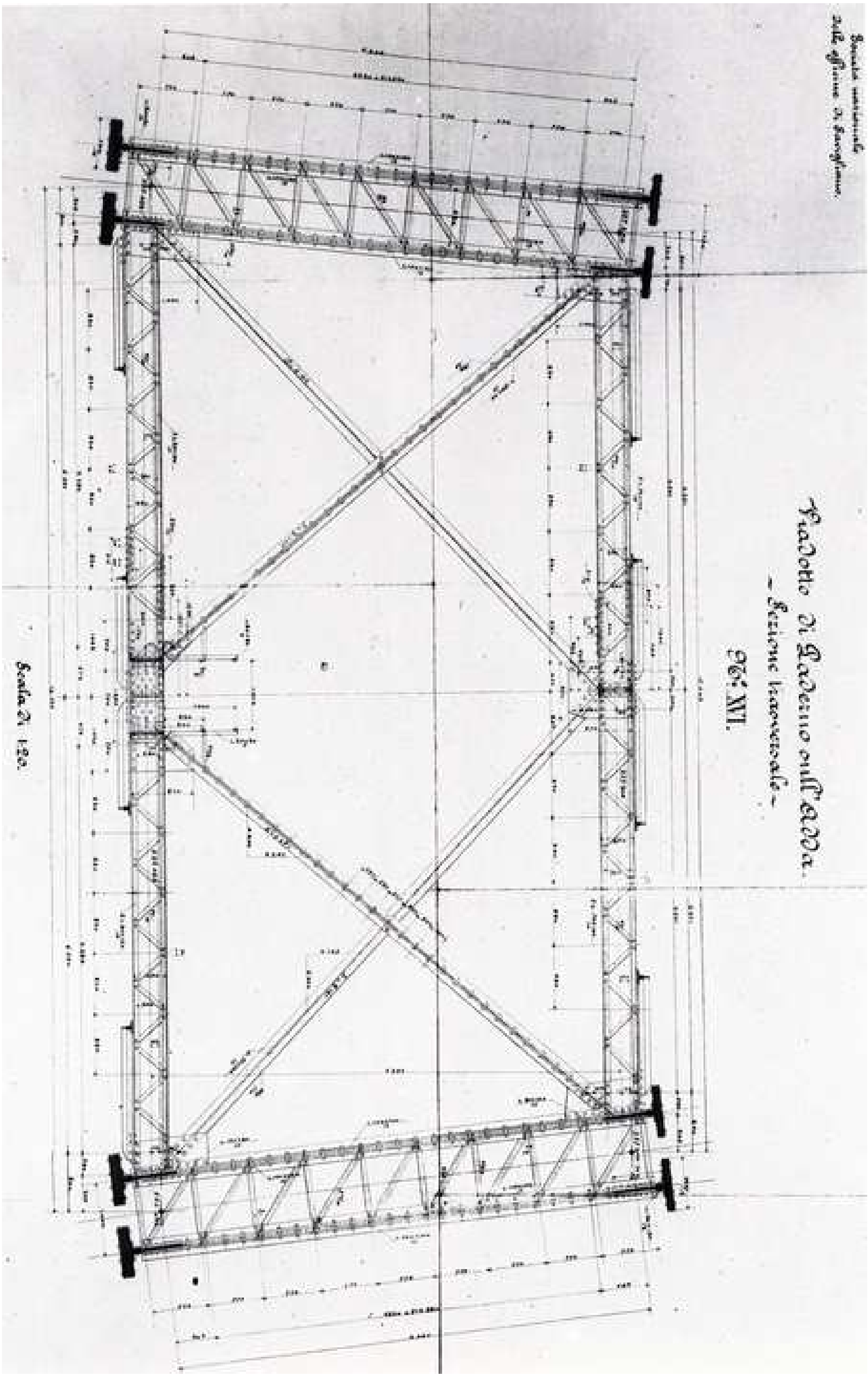

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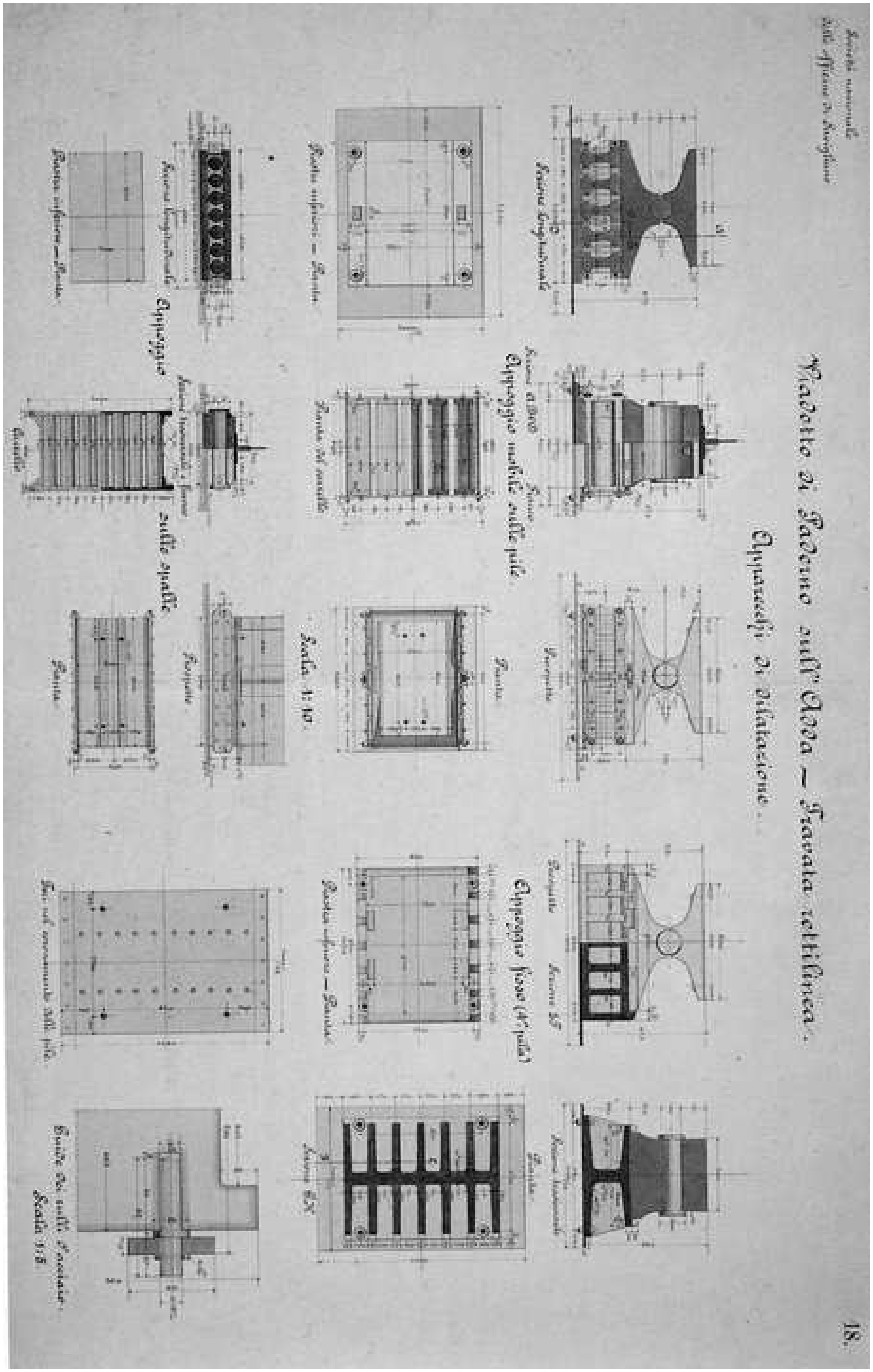

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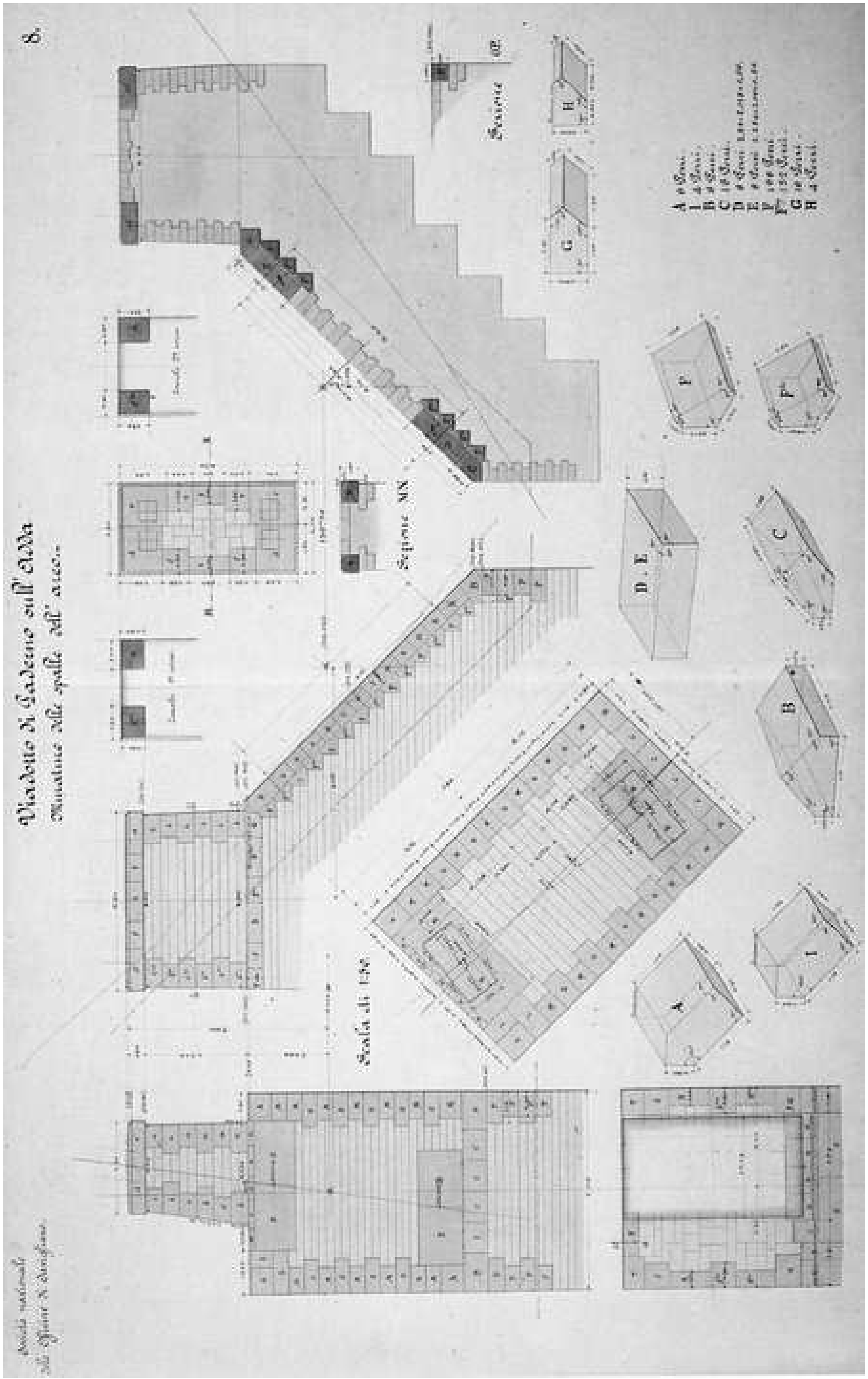

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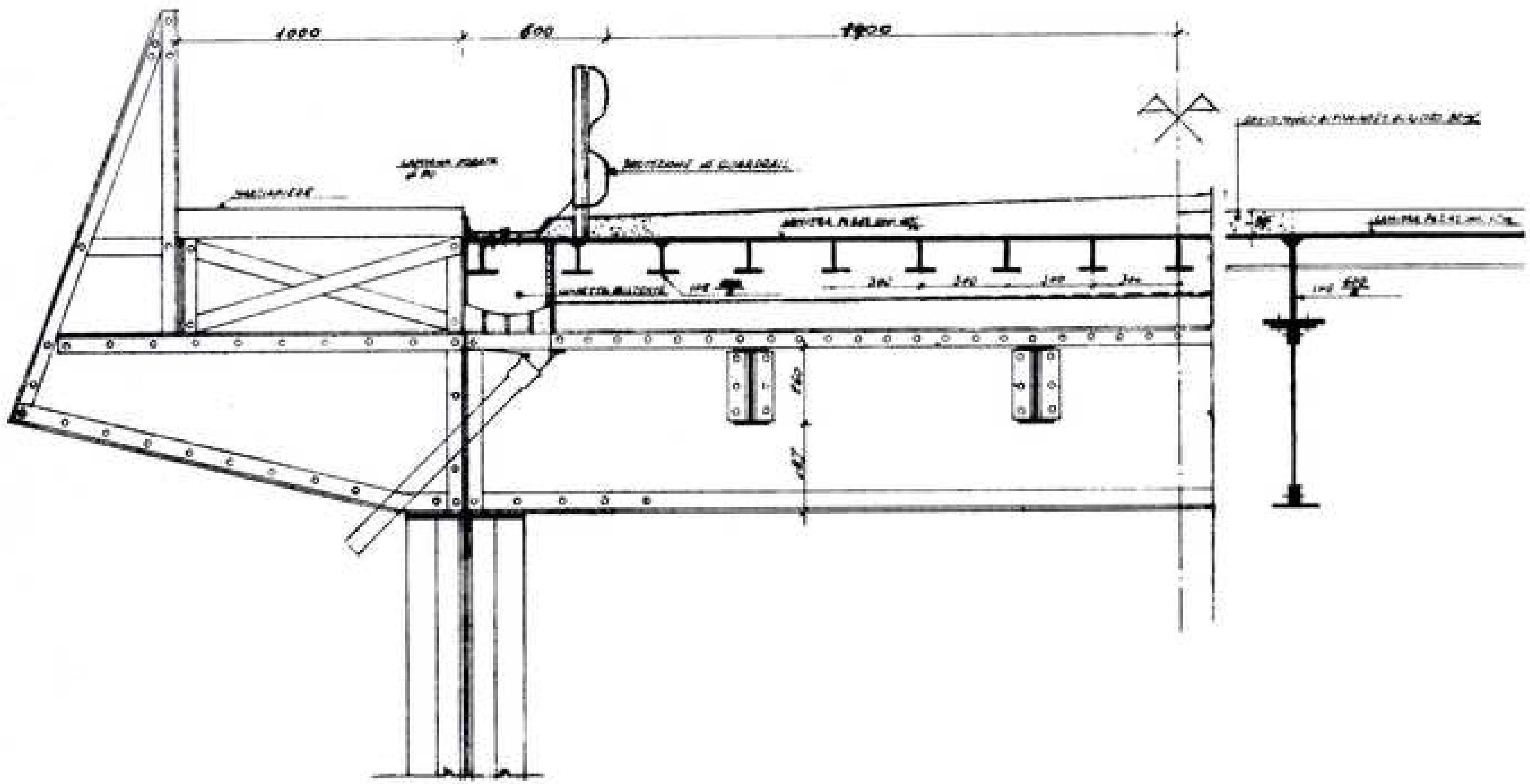

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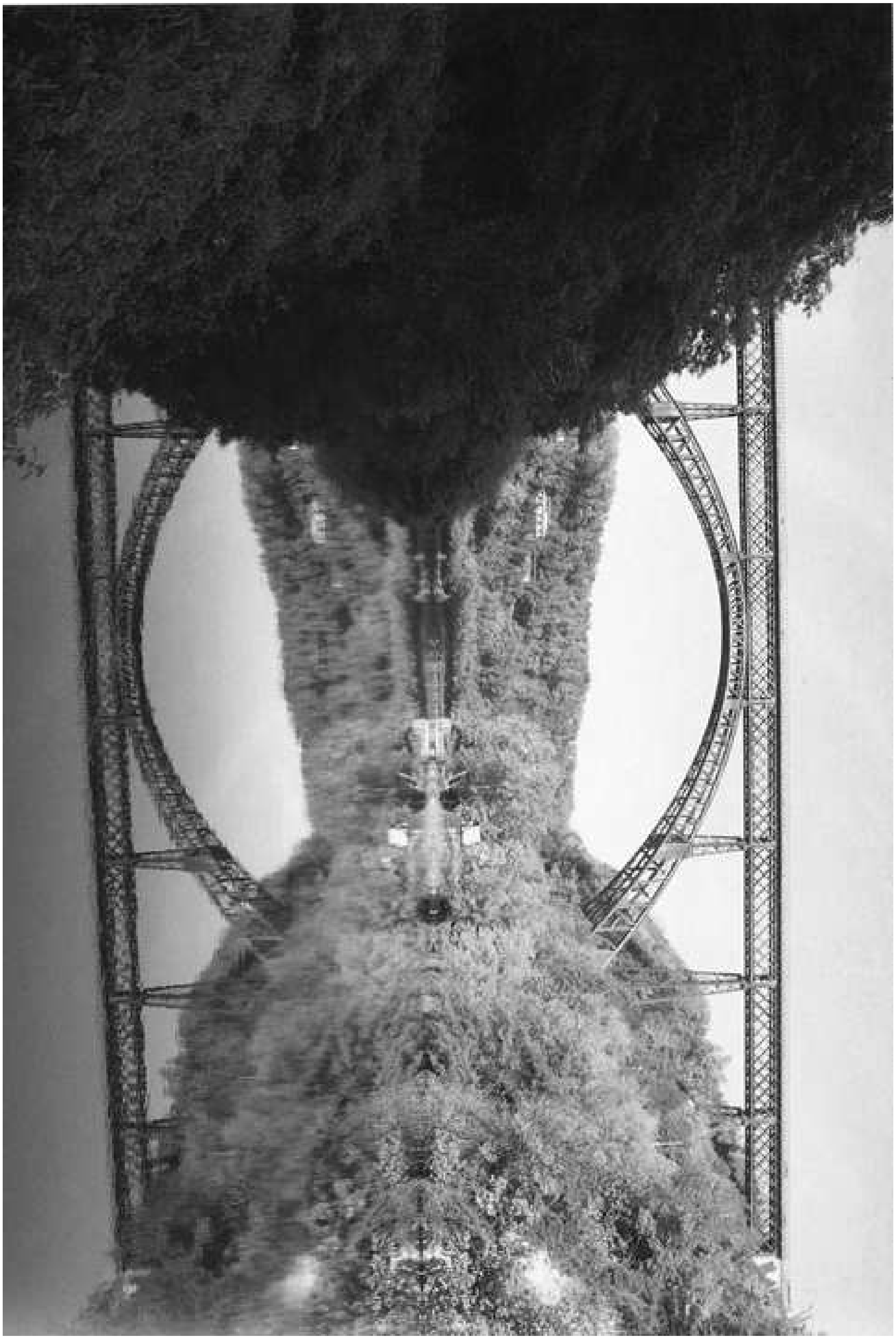

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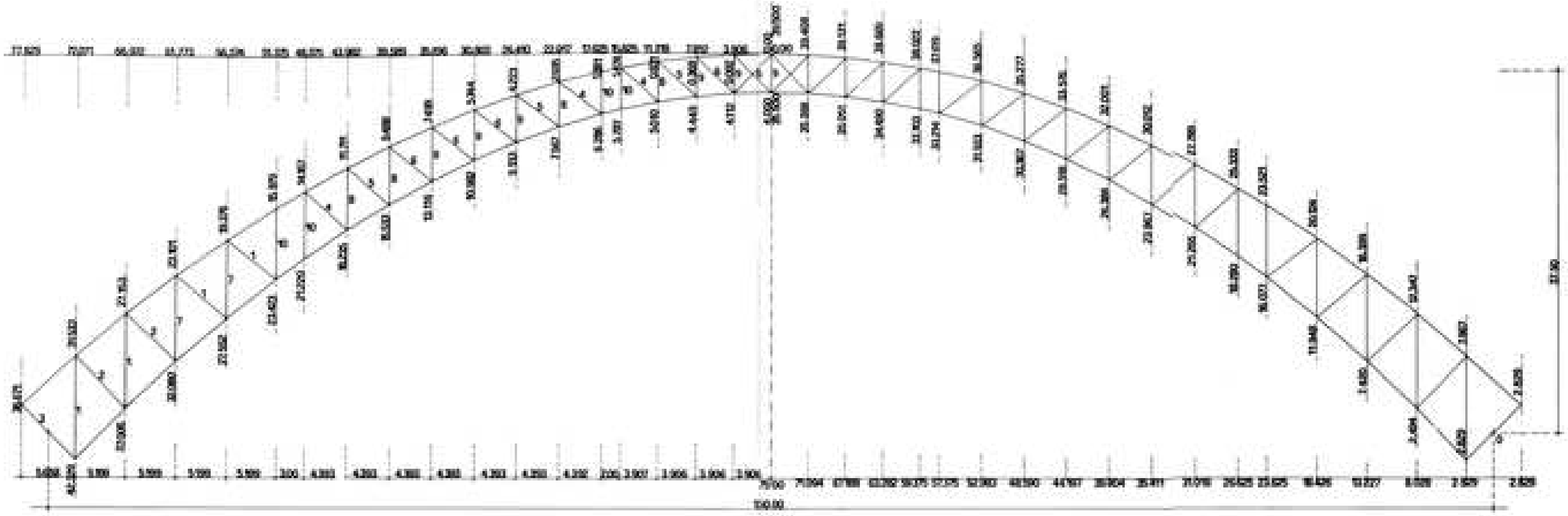
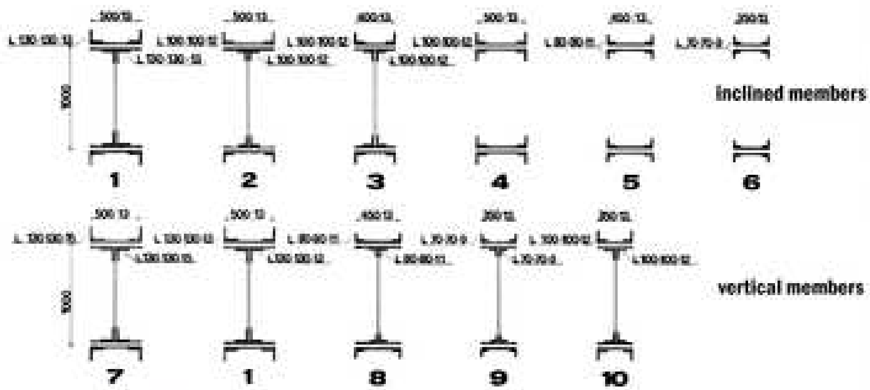
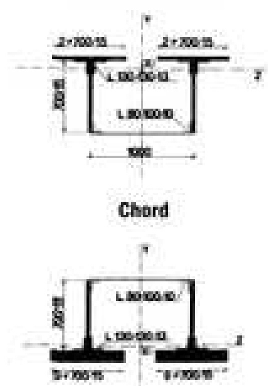


Arch coordinate - 1:200

Arch members - 1:10





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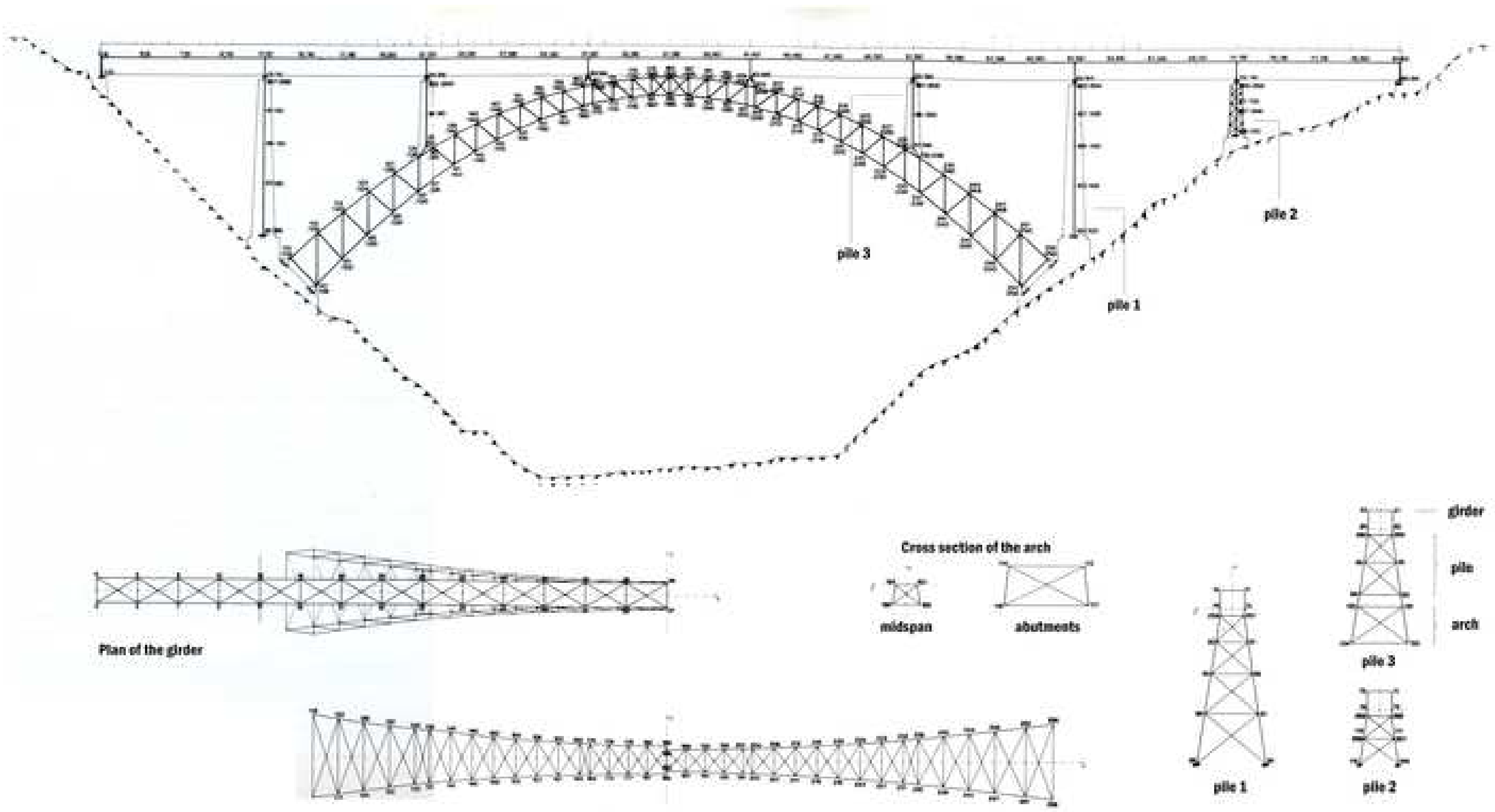

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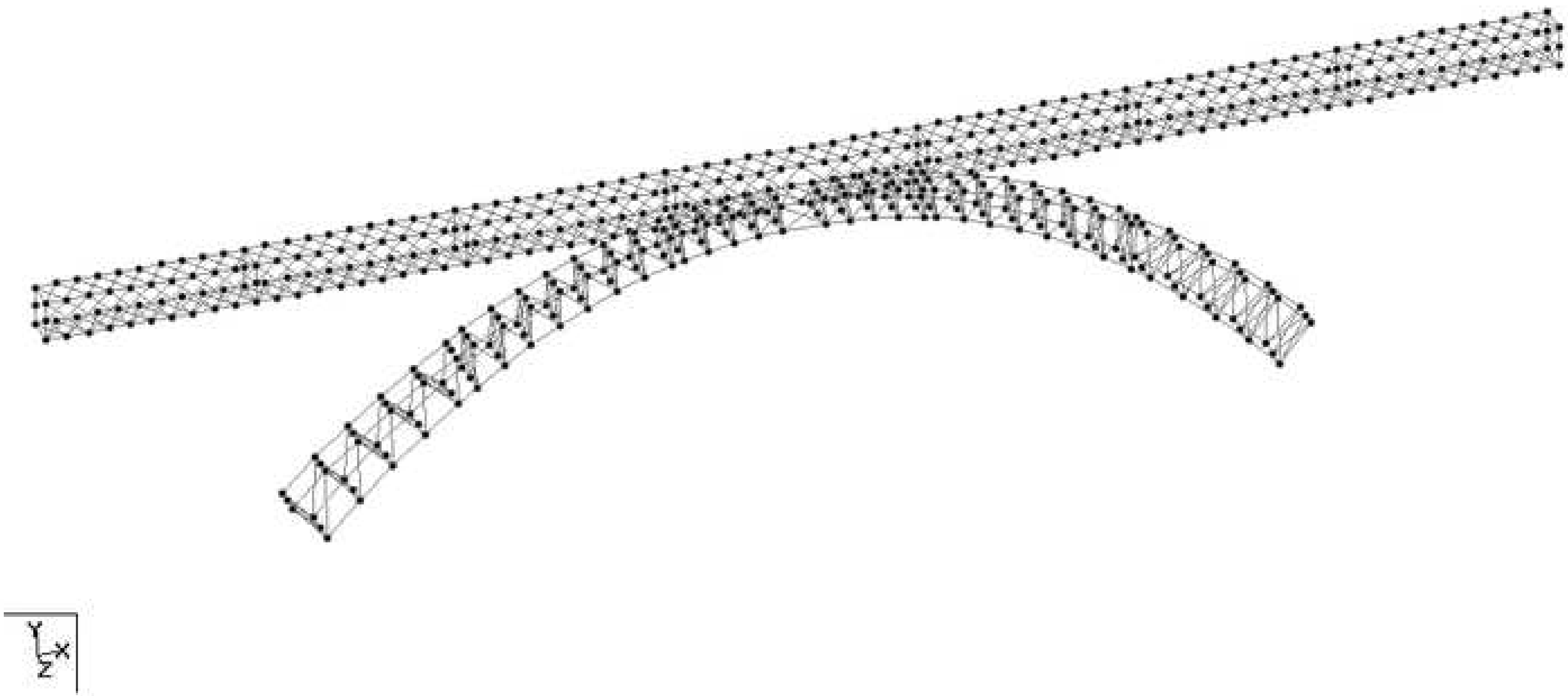

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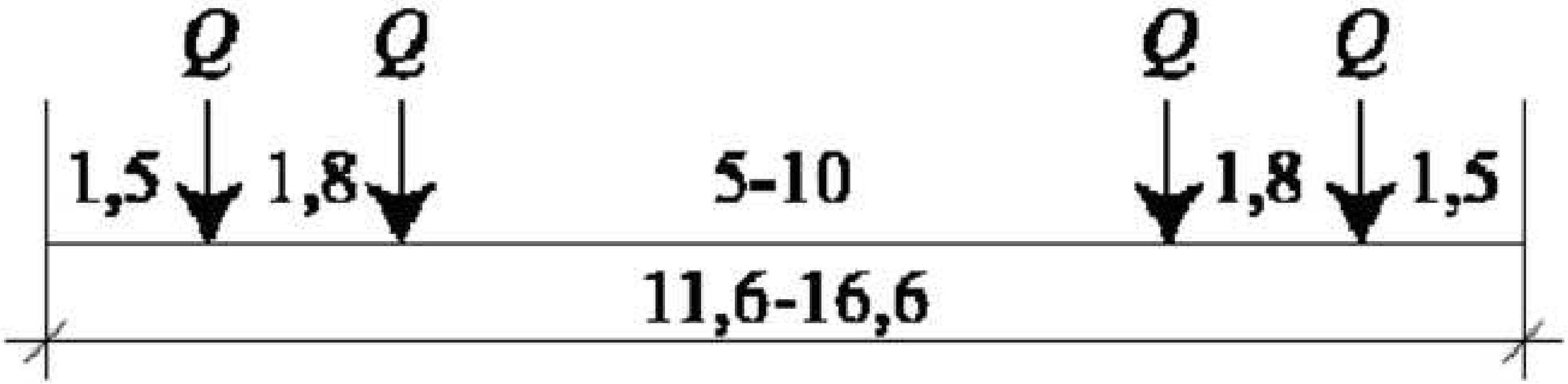

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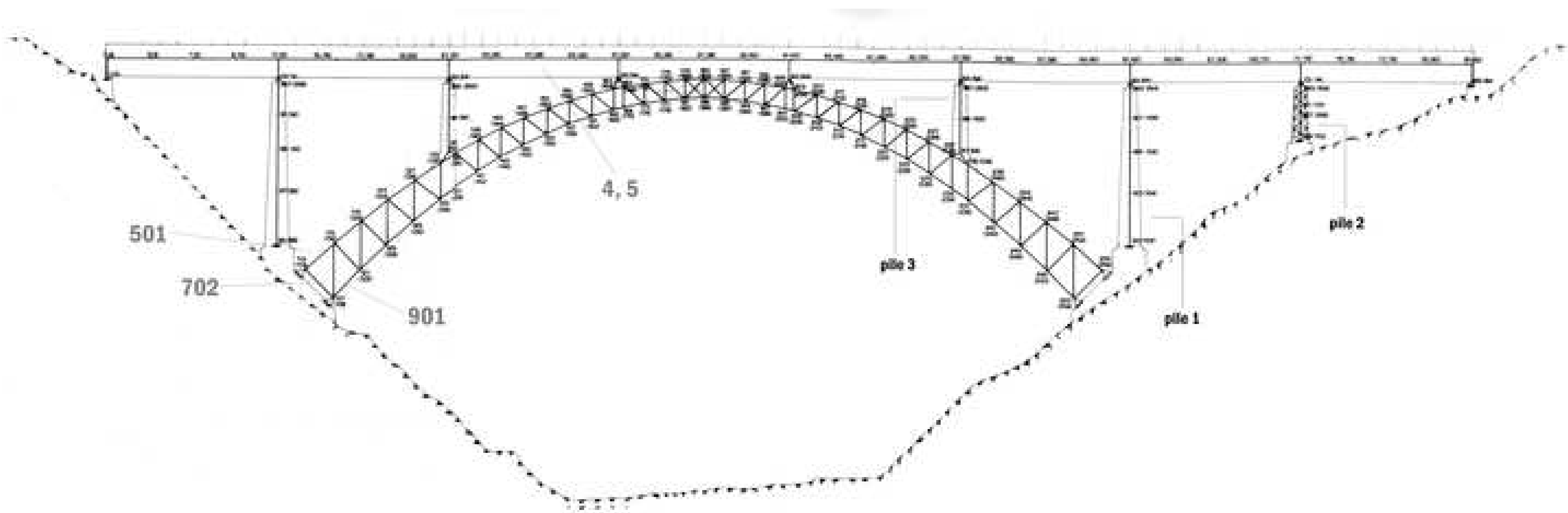

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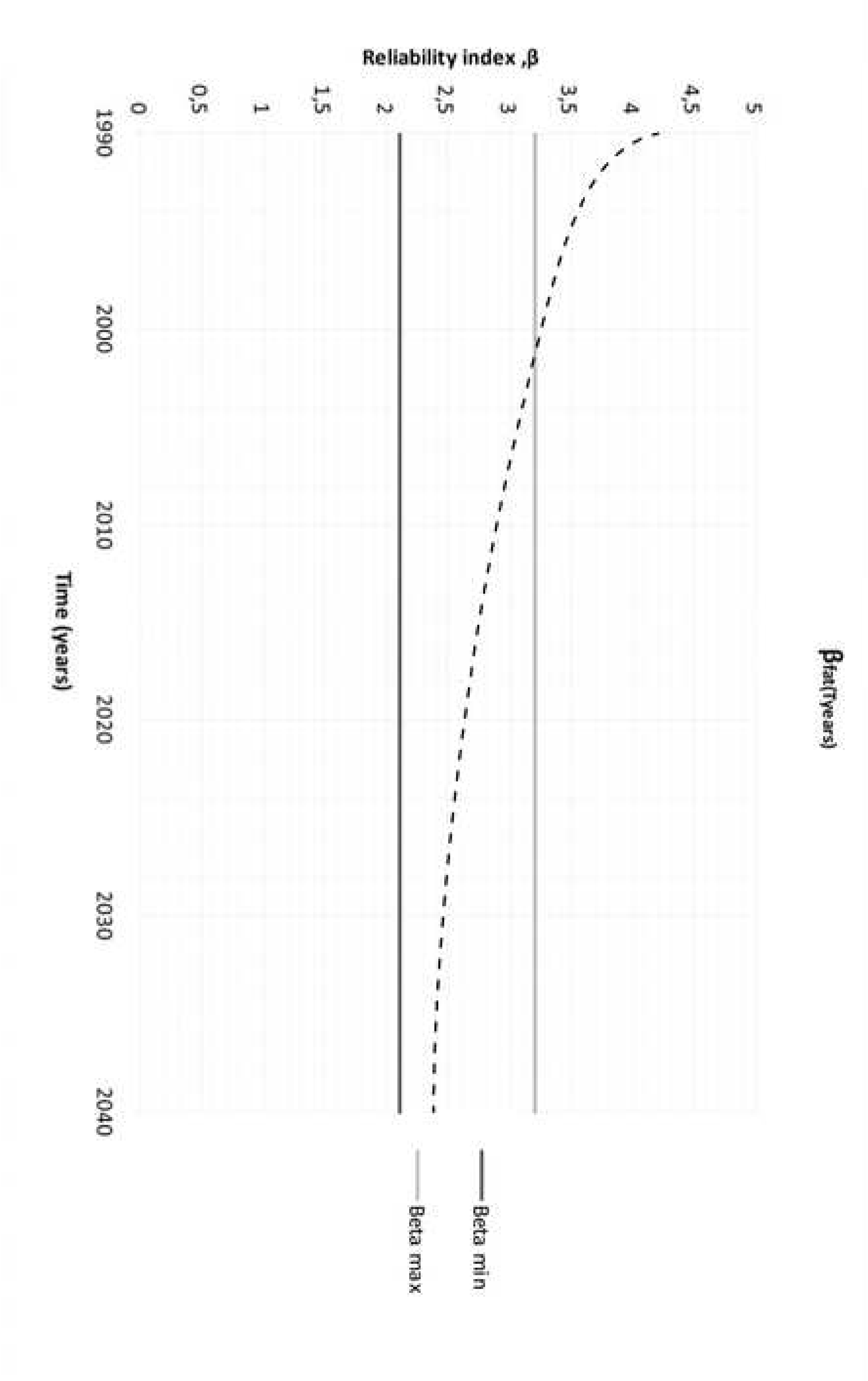

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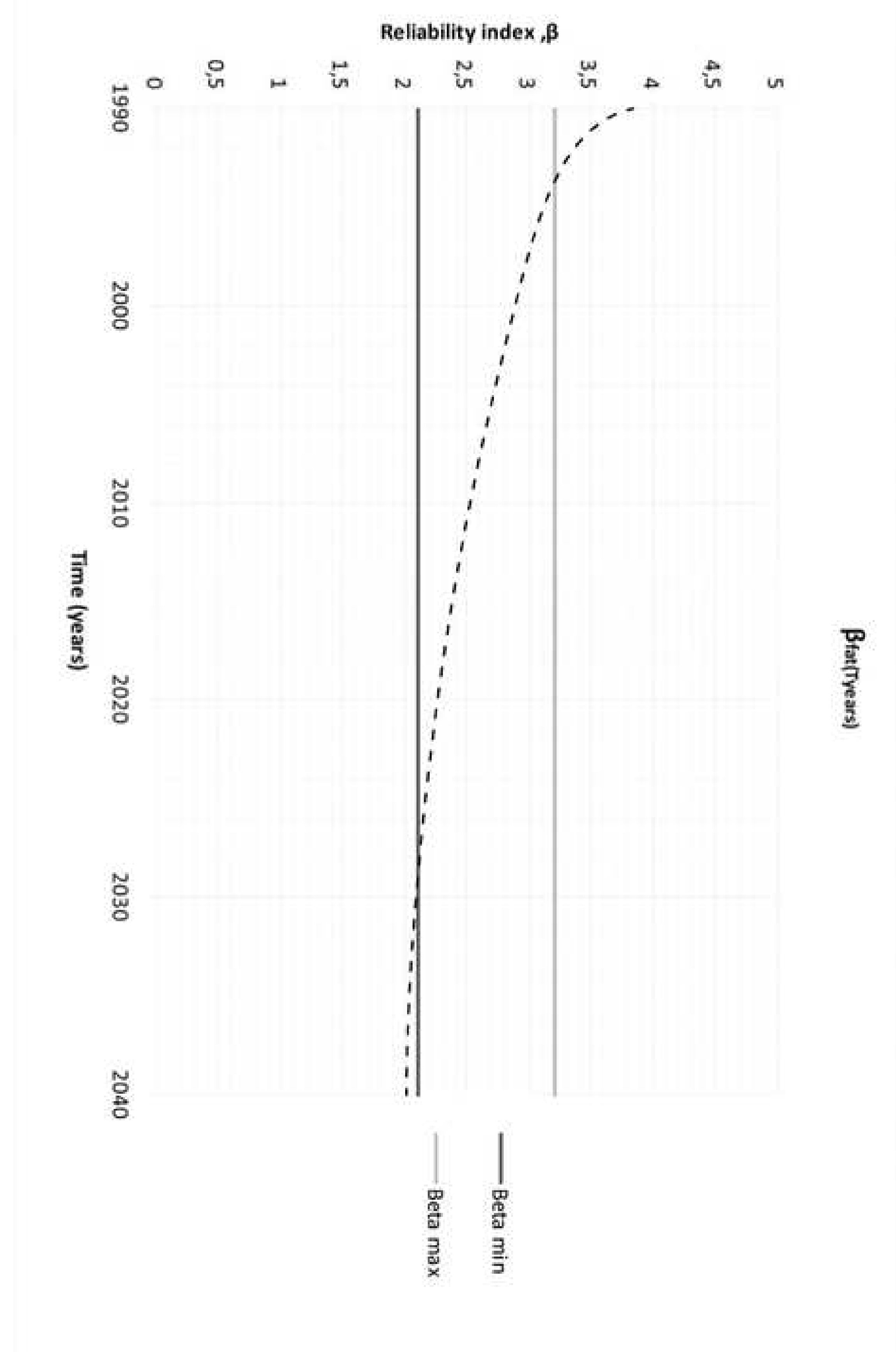

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