HIGH STRENGTH STEELS FOR LAUNCHING BRIDGES STRUCTURES

Emanuele Maiorana – OMBA Impianti & Engineering S.p.A., Italy
Alberto Miazzon – OMBA Impianti & Engineering S.p.A., Italy

ABSTRACT

Starting from the description of an important realization among the Italian orthotropic deck bridges, the Verrand viaduct, the discussion deals with a particular situation that has been resolved by the employment of High Strength Steel. The launching execution of the viaduct, in fact, required for the particular planimetric and altimetric plans of the area interested by the highway, the use of special equipments and large employment of high yield limit steels S690. The solution used was the best compromise between the actual technical state of art in such a steel structure and the final result, taking obviously in account the technical and economical aspects. These aspects are clearly discussed in the text to focus when and where it was the best compromise and there are given some indications to know why chose a high or super high strength steel instead of a standard steel type.

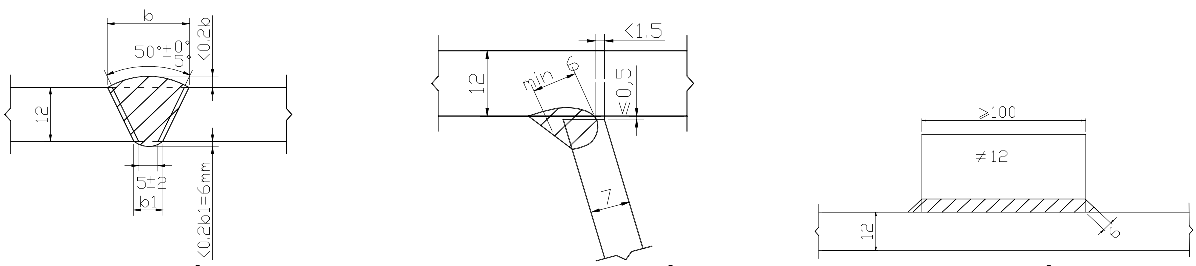
KEYWORDS

Launching bridge, Verrand viaduct, launching experiences in Italy, S690, state of art.

INTRODUCTION

To improve the mechanical properties of structural steel it is possible to follow two ways:
1) modify the chemical composition;
2) make a thermo-mechanical treatment during the rolling.

The second way is the most interesting in particular dealing with the welded structures, because the first way leads to an increase of equivalent carbon value (CEQ), value that is an index of weldability (to a CEQ value increasing corresponds a rising of welding difficulty in order to perform a correct welded joints). Although with the actual steels production procedures of S420 and S460 grades, the cost respect the S355 grade is not much greater, the use of these first steels isn't full justify in bridge constructions. This happens because a bridge is subjected to heavy cyclic loads and welded details are designed taking in account the Wöhler curves that are the some for of all the low allow steels. The Wöhler curves in fact give the Δσ for each welded detail in a steel structure, like a bridge, depending of the number of fatigue cycle (generally 2millions of cycles). See Fig.1 some details.



$\Delta\sigma$ = 80 N/mm^2 Transverse butt weld

$\Delta\sigma$ = 71 N/mm^2 Orthotropic deck-closed stringers

$\Delta\sigma$ = 63 N/mm^2 Weld attachments and stiffeners

Fig. 1 Examples of welded details in a bridge [1]

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The increased limits of yield in the up-graded steels (H.S.S.) aren't useful in the design although by the use of H.S.S. allows the element thickness reduction.
At the other hand, the use of High Strength Steels are interesting to design elements or structures, where the fatigue isn't important, provided that the check for the instability is satisfied. The result is the reduction of the material quantity.
As said, in many cases about the bridges structures it isn't helpful to use steel with elastic limit greater that of S355 ($f_y \geq 355$ N/mm^2). Generally, the grades of S355 in the bridge applications are S355J2G3 and S355K2G3, depending on the thickness of the elements and the temperature of the environment on which the structure is built. Sometimes, for length spans or very wide section are helpfully used thermo-mechanical types like S460 ($f_y \geq 460$ N/mm^2) or like S690 ($f_y \geq 690$ N/mm^2).
The consideration developed herein for the steel type used isn't comprehensive of particular loads configuration, like exceptionally earthquake forces or great railway or roadway loads that could acting only few times in the service life of a structure but if taken in account in the design could leads to extra dimensions not useful for the standard loads. In this last case the employment of High Strength Steels could be helpful for the optimization of economical and technical aspects with the full reaching of strength and fatigue limits of the elements product with these steels.

1. THE PUSH LAUNCH METHOD

The push launch method, where the bottom assembly and erecting is not possible, it is without doubts the most used and interesting, specifically for the girder bridges like concrete-steel composite bridges or orthotropic deck bridges.
This technique is normally employed for continuous girder bridges with length not greater than 160 m but can be used with appropriate modifications also for simple supported girders.
Otherwise it is necessary that some geometrical conditions will be satisfied:

- bridge alignment must be straight-lined or constant circular ray;
- transversal slope must be constant or gradually variable;
- viaduct section must be of constant high (the bridge intrados must be straight-lined);
- the yard must have a suitable assembly area of the sections in line with the alignment, at least behind the abutment of launching.

The concrete-steel composite bridges takes securely advantage in the employment of the longitudinal launching technique for which, like in the cantilevered technique, the steel structure dead loads of the structure during the erecting phase is a significant parameter. Since the structure during launching must over pass the spans with a cantilever scheme and the girder elements passing on the piers are subjected to greater negative bending moment while they are designed to resist at positive moment in service life, some artifices must be adopted to reduce the forces during the erection, as the following ones:

- using temporary supports at midspans, to reduce the length of the main spans in launching phase so reduce the cantilever beam length;
- employing a lattice structure more light of the bridge structure to reduce the forces in the bridge girder elements section passing over the piers (Fig.2). The length of the lattice girder is established by the main spans to over pass and, as said, by having greater negative bending moment during the launching phase in the some sections as in the service life. It is important to use a lattice girder with a curve intrados to permit the correct approach of this in the sleighes;
- adopting a system of temporary stays into the bridge structure for the some reasons of the first point;

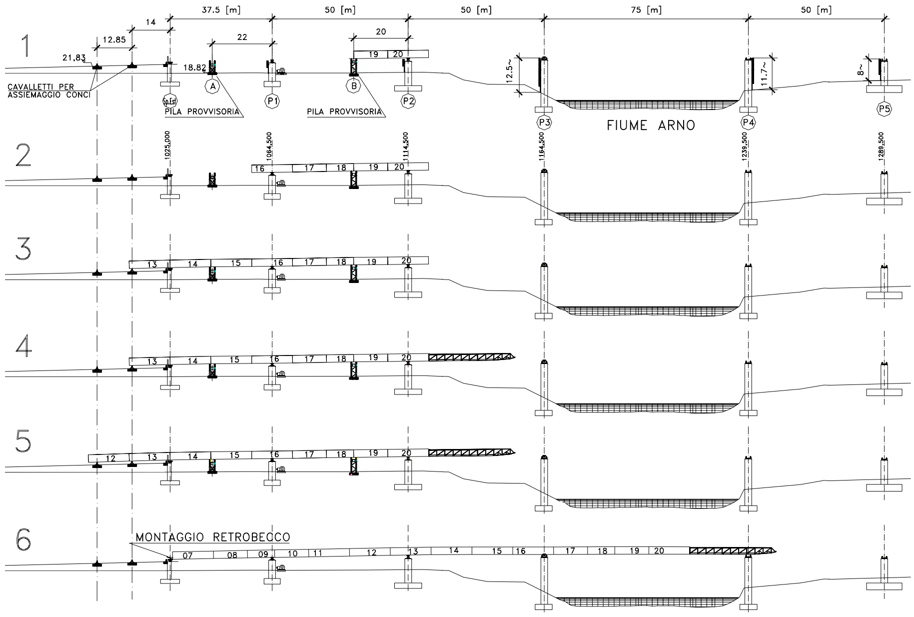

Fig.2 Lauching phases of a bridge above the Arno river (2001)

- strengthen the upper flanges where these aren’t adapt to satisfy the verifications during launching phase: this request generally happens in steel-concrete composite bridges, lacking the contribution of the slab to the stability of the upper flanges;
- place small girders under the lower flanges, for the all length of the main girders in the case in which the lower flanges and web panels aren’t verify to resist at localised loads growing from the support surfaces (sleighes). This situation happens using lattice girders.

It is now clear that, case by case, can be adopted combinations of above solutions but it’s important to know that in the preliminary design must be taken in account the assembly and launching phases and the yard requests (Fig.3 and Fig.4)

Fig.3 Arno viaduct (2001). Launching phase

Fig.4 Calitri viaduct (2000). Launching phase

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2. NUMERICAL APPLICATION

The employment of High Strength Steels can be useful in the design of launching equipments to over pass long spans when it isn't possible the bottom assembly and erecting technique. Among the launching equipments, one of the most important is the lattice girder.
The aim of this section is to explain the main phases of lattice girder design used in longitudinal launching of steel bridges. Although it isn't possible to give general rules to reach this objective, for the great number of parameters involved in the problem, herein are assumed some hypothesis by the experience of many years of construction with this technique. The conclusion of the discussion gives the idea of the grade steel type necessary for the construction of a lattice girder in order to reduce the steel structure dead load and assure the integrity of the bridge girders during the erecting without not useful increase of elements thickness respect the service life loads.
In the numerical application it is considered a concrete-steel composite bridge for which, in launching phase, is acting only the steel structure dead loads of the steel girders, while in the service life the forces are given also by the concrete dead load (slab) and by the traffic load (vehicles). So, the main loads acting in a bridge during the service life are usually the following:

- steel structure dead load g_1
- concrete dead load (slab and guard rails) g_2
- traffic load (vehicles) q_1

In order to know the contribution of each load for the design of the main girders sections so to know the stress reserve that can be helpful in the launching phase respect the service life, the ratio between the dead loads and steel structure dead loads and between the traffic loads and steel structure dead loads are calculated.
For the bridge Italian code [2], the traffic loads are identified by three concentrate loads P = 200 kN (distance from each other a = 1.5 m) and two uniform loads q = 30 kN/m of length c, until the supports (the distance from the lateral concentrate load is b = 6 m). See Fig.5.

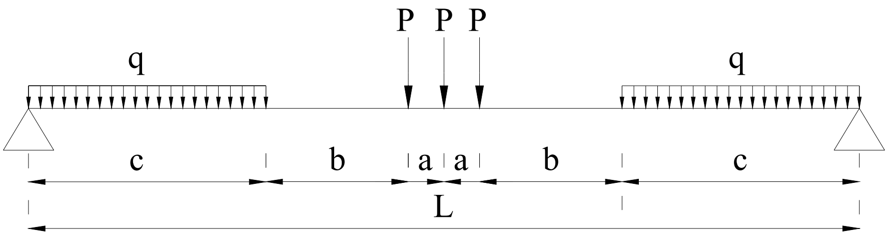

Fig.5 Traffic loads configuration for bridges [2]

A simple beam on two supports is considered, symmetrically loaded; from the equilibrium equation, the maximum bending moment is at the midspan, calculated with Eqn.1:

$$M_{L/2} = \frac{3PL}{4} + \frac{qc^2}{2} - Pa \qquad (1)$$

The uniform equivalent load equals to the configuration in Fig.5 is found with Eqn.2:

$$q_{1,eq} = \frac{M_{L/2}8}{L^2} = \frac{4350}{L^2} + \frac{300}{L} + 30 \qquad (2)$$

In Table 1 are reported some values of equivalent traffic load for various length of light. A graphic reports the L - $q_{1,eq}$ relationship.

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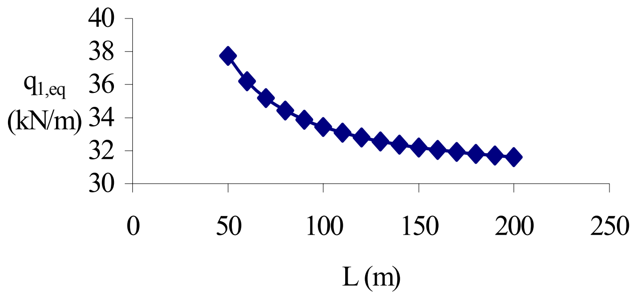
Tab.1 L - $q_{1,eq}$ relationship

L (m)	$q_{1,eq}$ (kN/m)	L (m)	$q_{1,eq}$ (kN/m)
50	37.740	130	32.565
60	36.208	140	32.365
70	35.173	150	32.193
80	34.430	160	32.045
90	33.870	170	31.915
100	33.435	180	31.801
110	33.087	190	31.699
120	32.802	200	31.609



Herein are reported few considerations of a first analysis of a longitudinal launching bridge technique, with the aim to arrive at general evaluations starting from a representative particular case.
A parametric study is done to permit the erection of a bridge without not useful over dimensions respect the design for service life or using temporary equipments (except the lattice girder).
Taken in consideration a bridge on four supports, three spans (the main of length L = 100 m and the later ones L = 70 m), two main symmetrical I beams with the width slab of 10 m (the transversal section is in accordance of the Bredt theory). In the considerations developed it is considered, for the symmetry, half width. The traffic load is referred to the span length of 100 m, minor than that of 70 m, so it is more restrictive in the analysis. By the Table 1, the equivalent load is q_{eq} = 33 kN/m.
Considering two lines of traffic load, one at 100% and the second at 50%, the final load is $q_{eq,tot}$ = 1.5 x 33 = 50 kN/m, for each girder 25 kN/m.
The dead loads are due from the concrete slab (thk = 250 mm, width = 10 m; γ = 25 kN/m^3), from the road-plane (thk = 100 mm, width = 7 m; γ = 20 kN/m^3), from two foot-pathes (thk = 450 mm, width = 1500 mm; γ = 25 kN/m^3) and from two guard rails (1.5 kN/m).
g_2 = 0.25x10x25+0.10x7x20+2x0.45x1.5x25+2x1.5 = 113.25 kN/m; for each girder 56.63 kN/m.
The allowable tension is 240 N/mm^2 and it is reached on each beam with a steel structure dead load section of about 8 kN/m.

g_1 = 8 kN/m	or 9%	that gives a tension of	22 N/mm^2
g_2 = 57 kN/m	or 63%	that gives a tension of	151 N/mm^2
q_1 = 25 kN/m	or 28%	that gives a tension of	67 N/mm^2

In Fig.6 is reported the girder sections (max length 22 m) for the optimization of the section dimensions.

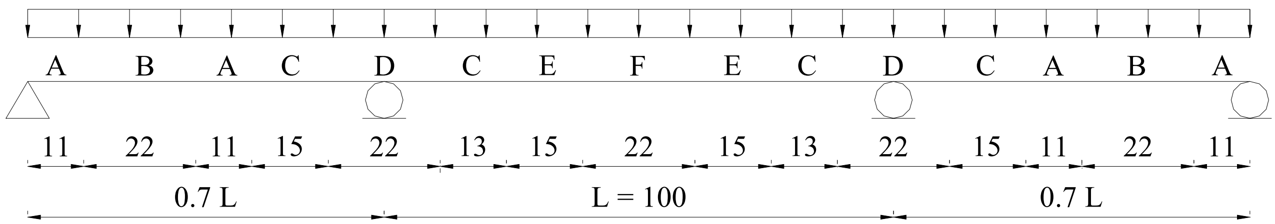


Fig.6 Girder and sections

In Table 2 are reported for each girder section the bending moment due to each type of applied load. The girder is designed for the combination of the steel structure dead load, the concrete dead load and the traffic load, not including the minor loads and the coefficients applied to each load for each of the five types of combinations as reported in the Italian bridge code [2].

Tab.2 Sections and bending moments for each load conditions

sec	flanges (mm)	web (mm)	Mg_1 (kNm)	Mg_2 (kNm)	Mq_1 (kNm)	M_{max} (kNm)
A	500 x 25	3000 x 12	1446	10306	4746	16498
B	530 x 30	3000 x 12	1925	13715	6189	21829
C	850 x 35	3000 x 16	3569	25433	10690	39692
D	1240 x 55	3000 x 20	7315	52120	22308	81743
E	680 x 30	3000 x 12	2201	15681	8942	26824
F	730 x 30	3000 x 12	2685	19130	7430	29245

The design of the structures according to the life service must be controlled in order to assure the stability of the girder during the launching phases. This phases are analyzed by one-dimension models and by two-dimensions models for the exact evaluation of the most heavy forces for each element in this relevant and also short period situation. In Fig.7 is showed the final configuration of girder on four supports (L11) and some possible launching configurations (L1-L10) consisting in a beam on two supports plus a cantilever beam from A2 to A3. The lattice girder is not included in the figure.

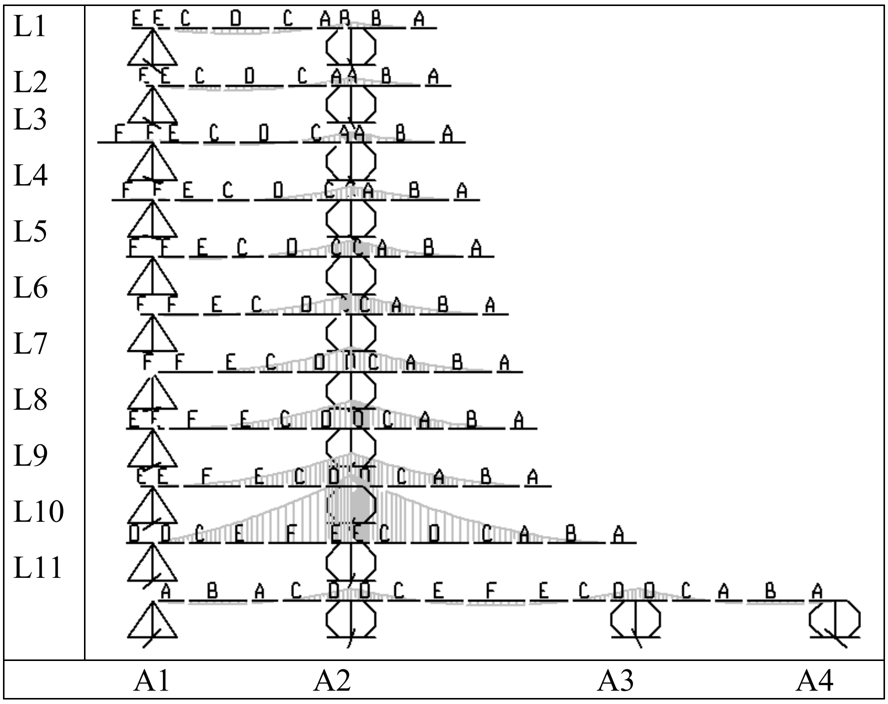


Fig.7 One-dimension models of different length of girder from A2 to A1

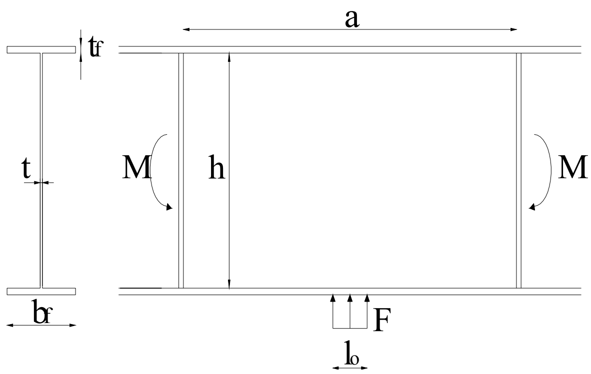
Doing a comparison between the negative bending moment in A2 section for different length of the cantilever beam from A2 to A3, it is clear that the forces from L1 to L10 grow in the elements passing over the A2 support (Table 3); the forces must be report to the service life of the girder elements.

The main problem is that the elements passing over the support A2 are subjected to a combination of: a concentrated load due to the pier reaction to the steel structure dead load and to a correspondent negative bending moment. From the combination of the concentrated load and the bending moment, problem known as patch load, it is derived the difficulty of the bridge verification in order to assure the safety ratios during the launching phases (Fig.8).

Tab.3 $M_{max} – M_{A2}$ relationship

	sec	L_{av} (m)	M_{A2} (kNm)	M_{max}/M_{A2}		sec	L_l (m)	M_{A2} (kNm)	M_{max}/M_{A2}
L1	B	70	3600	6.06	L6	C	45	12100	3.28
L2	A	65	4900	3.37	L7	D	40	14400	5.68
L3	A	60	6400	2.58	L8	D	35	16900	4.83
L4	C	55	8100	4.90	L9	D	30	19600	4.17
L5	C	50	10000	3.97	L10	E	-	40000	0.67

Fig.8 Patch load f-L



The worst situation is verified when the ration between the maximal bending moment in exercise and during the launching phase regard the girder elements designed for positive bending moment while in launching phase they are subjected to a negative bending moment. It's chosen the configuration with a lattice girder of length equal to or greater than 60 m, specifically, by the experience, good results can be achieve with a length of 2/3 the main length span to over pass.
In a second phase are employed the two-dimensions models (Fig.9). A first group (a) regards the maximum cantilever beam length, the most critical situation for the section in A2, while a second group (b) regards the most critical configuration for the lattice girder elements (patch load).
Herein, for each group are considered three types of lattice girder; the distance between upper and lower beams is 3 m as the main beams high while the lateral truss elements are spaced from 4 m, 3 m and 2 m (respectively noted with $_{1,2,3}$)

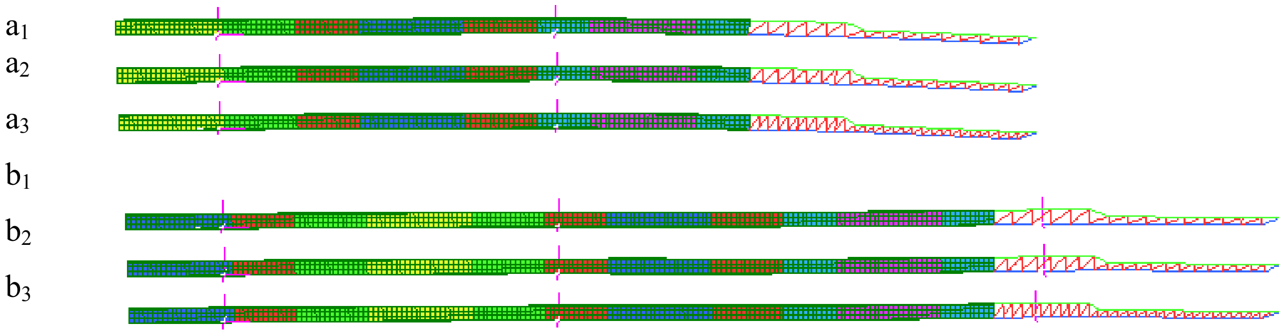


Fig.9 Two-dimensions models to verify the girder for the maximum cantilever length (a) and maximal reaction for the lattice girder (b)

The calculation of the girder during the launching phase is satisfy if the configuration (a) present a positive check for the patch load for each element in A2 (Eqn.3) [3]

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$$\frac{F_{Sd}}{R_{a.Rd}} + \frac{M_{Sd}}{M_{c.Rd}} \leq 1.5$$

(3)

In the patch load verification of the girder element passing in A2, the web panel of the girder is divided into sub panels with the employment of longitudinal stiffeners (the transversal stiffeners are distanced from each to other of a = 3 m). Since the ratio between the panel thickness (t = 12 mm) and the distance b from the lower flange (on which acts the concentrate load) to the longitudinal stiffener must be not minor that about $\lambda = b/t = 43$ (value that assure the full yield strength of the web steel instead of the collapse for instability phenomena), so the aspect ratio is about $\alpha = a/b =$ 6.5.

In Table 4 is given the allowable tensions for each type of steel that can be used in the construction of the lattice girder. It must be considered also that the reduction of the section element by the increase of the tension using superior grades of steel, is less that considered in a first time because of the necessity to have a minimum thickness of the elements for the instability problems that could be verify (the factor is $\sqrt{A}$; where A is the area ratio). In Fig.10 is reported a bars diagram in which is underlined that for long span length (≧ 100 m), the material that must be used for lattice girder is a H.S.S. in order to guarantee the respect of the European code [3] security ratio (1.5).

It is also clear that changing the design parameters, different conclusion can be reaches, although for medium or high span length, the use of H.S.S. seems very helpful in the construction of launching structures, like the lattice girders.

Tab.4 Steels, yield limits and costs

Steel	f_y (N/mm^2)	σ_{amm} (N/mm^2)	$\sigma_{amm,i}/\sigma_{amm,S355}$	$cost_{Si}$ [May 2005] (€/t)	$cost_{Si}$ / $cost_{S355}$
S355	355	240	1.00	650	1.00
S460	460	310	1.30	1000	1.54
S690	690	460	1.90	1500	2.31

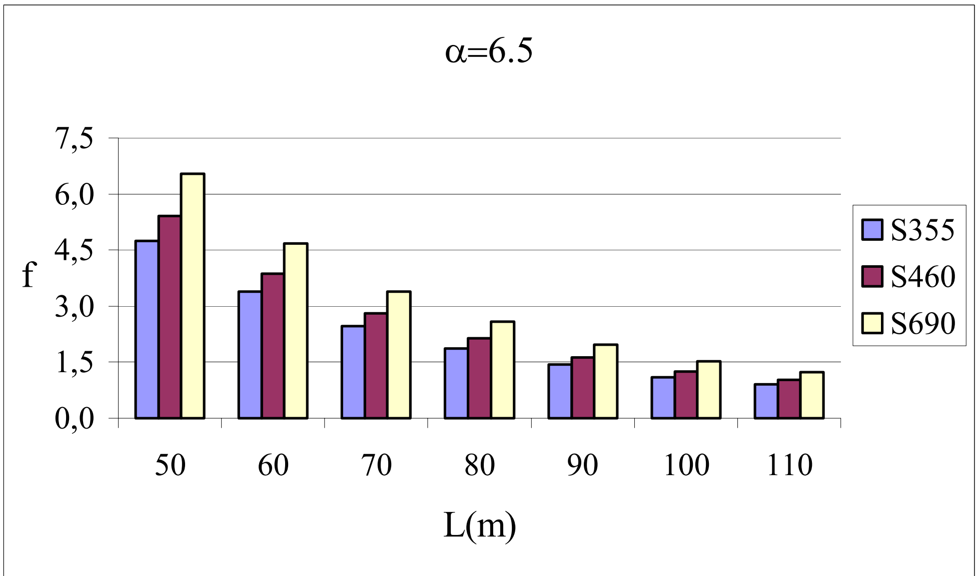

Fig.10 f-L relationship

3. AN EXPERIENCE: THE VERRAND VIADUCT

The Verrand viaduct is an orthotropic deck bridge, part of the Mont Blanc-Aosta highway, positioned in the 3th lot, Mont Blanc Tunnel-Morgex. The viaduct is located at Prè Saint Didier (Aosta), near Courmayeur, beside of the existing S.S.26 (Mont Blanc Tunnel-Aosta national road) and it is necessary to overpass the valley between the national road and the Dora Baltea river. It was finished on August 2002 after 2 years of work and with use of total steel quantity of about 6100 t, the viaduct assures after the completion of the highway to go at the Tunnel with a completely highway road.

Fig.11 Connection of the bridge parts, built in two different yards (Aosta and Mont Blanc sides)

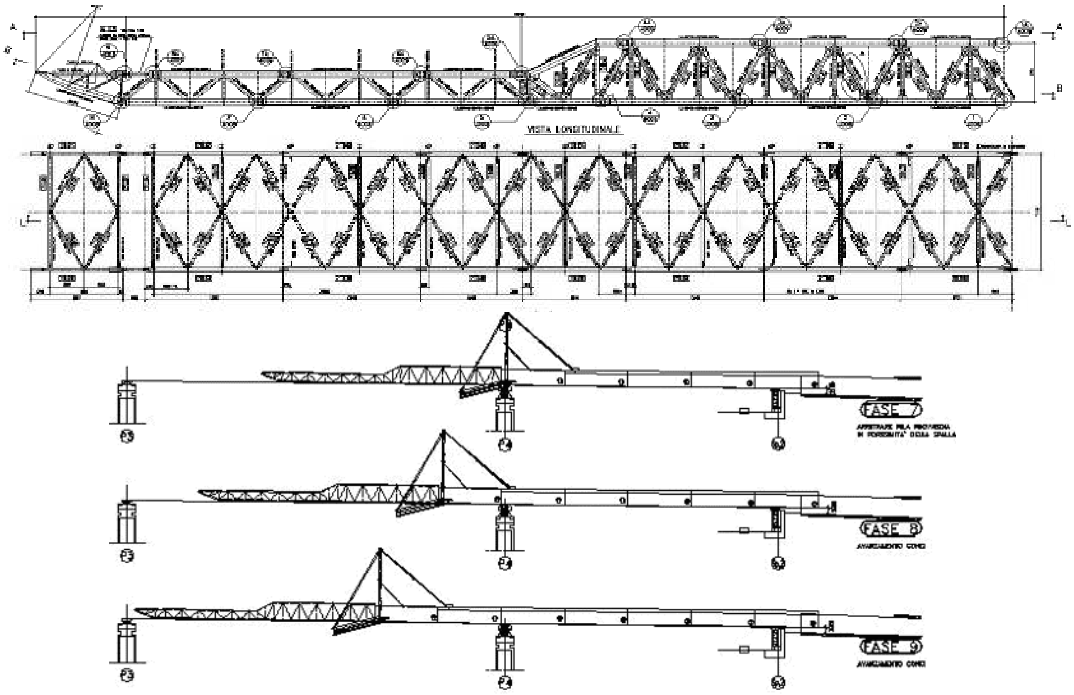

Fig.12 Lattice girder (S690). Lateral view and plan. Some launching phases

The possible structural alternatives were all characterized by the choice to realize an unique motorway viaduct for all the roadways, with consequent width nearly 20 m.

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For the motives regarding the site, the length of the intermediate spans mustn't be less than 100 m. It was decided for an orthotropic deck bridge, two principal beams and inferior bracing, of five spans, respectively, 97.5+135+135+135+97.5 m, with four intermediate piers.
The stresses for the section are calculated considering the close longitudinal section stiffeners (with their full sections) for the local and global stress verifications. The verifications, have been conducted according to the method of the allowable tensions, with reference the Italian code CNR UNI 10011 [4].
The structural steel employed is of the type with improved resistance to the atmospheric corrosion ("type COR-TEN"), correctly described in the UNI EN 10155 [5]; in relation to the strength to the low temperatures the degrees used are the followings:

S355J2G1W for the elements settled of thickness t $\leq$ 40 mm;
S355K2G1W for the elements settled of thickness t > 40 mm.

The bolts are all of HSFG type (10.9 screws +10 nuts [6]).
The bolted joints has been of friction greep keeping of a typical coefficient of friction greep equal to μ=0.3, as it regards the friction joints of the girder and shear type for joints for wind-bracing and diaphragm. Nevertheless also the shear type joints have been verified in order to perform also at friction greep for the fraction of forces due to traffic loads.
The deck hasn't anticorrosive treatment because the material used, both because the characteristics of the environment of the place where it is located that can be considered suitable for the process of formation of a layer of oxide resistant and lasting man to be arrested the process of rusting in the superficial layers of the steel, preserving from an advancement of the corrosive trial.
The sections in which the viaduct is divided have length equal to 22.5 m, than the final elements of length equal to 20 m and of total number of 27 for the full length of the viaduct equal to 600 m. The deck is prefabricated in shop in panels of length equal to the sections, and of width equal to 2.75-3 m, for typical nine panels for every section; the sections are composed, by the panels, by the diaphragm and by the inferior horizontal wind-bracing.
The assemblage of the panels in the yard is realized with joints: the joints of the plate, both longitudinal and transversal, the longitudinal and transversal joints of the web and the joints of the inferior main girders. One of the problem that were evaluated immediately studying the launching of the viaduct has been the choice of the constructive typology and the most proper length of the lattice girder to allow the main structure over pass such important span (135 m), in relation with the remarkable unit steel structure dead load (100 kN/m).
The lattice girder used has to be sufficiently light to be able to be supported by the main girders, in the condition of maximum cantilever scheme and strong enough to hold the maximum reaction on the piers top.
The lattice girder of variable heigh, has the following characteristics: material was H.S.S. type, length 85 m and weight equal about 3000 kN, made by pipe profiles, in steel S690, beams realized

by welded beams in steel S500 - S690, connections among the various elements by welded pin-hinged in steel S690.
The specific equipments, realized by H.S.S., has allowed the reduction of the weight and the design of the steel deck bridge with no changes for the launching phases respect the service life design.

Fig.12 Lattice launch girder using S690

Fig.13 Shop pre-assembling

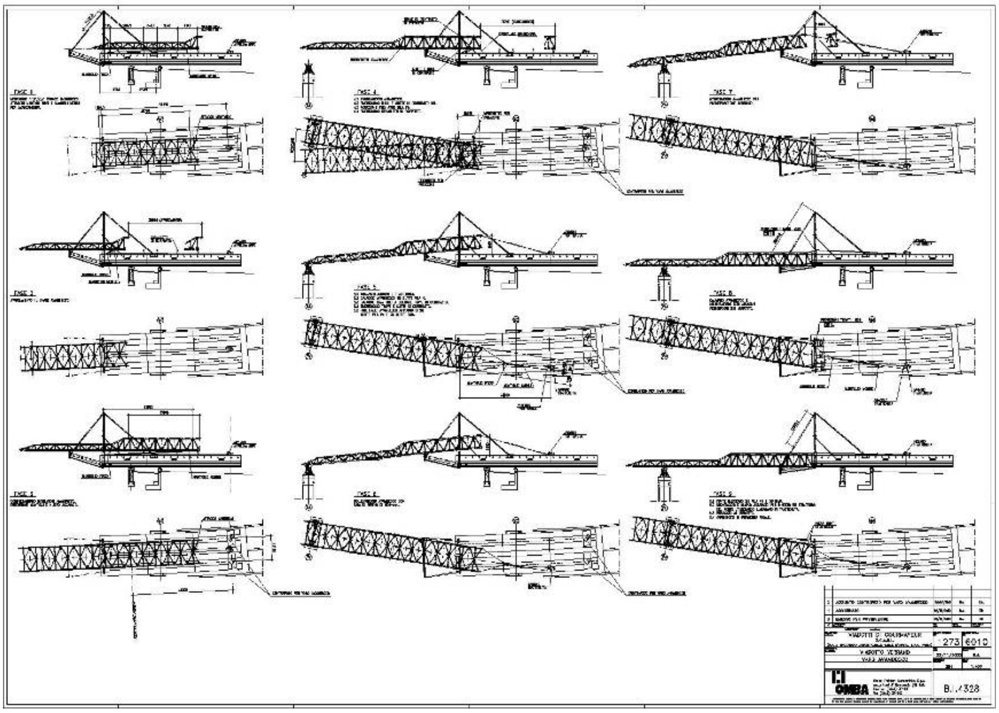

Fig.14 Lattice launch girder movements

Fig.15 Final test of the Verrand viaduct

4. CONCLUSION

With High Strength Steel the dimensions of the section and the thickness are reduced respect the same dimensions using carbon steel so the total weight of the structure is lower but to avoid the instability of compressed thin elements it is necessary to have a minimum value of thickness so the employment of the H.S.S. can be not always useful. The same considerations can be done for the fatigue detail, in fact always the problems involving cyclic loads regarding welded details isn't improved by the use of H.S.S..
Since the quantitative relationship between the stress range and number of stress cycles to fatigue failure, used for the fatigue assessment of a particular category of structural detail is the same for any kind of Carbon Steel grades and High Strength Steels have the same base chemical composition. So for structures that are subjected to fatigue, such as the bridges, the employments of high strength steels, usually, seems not interesting. Otherwise, these high yield limit steels can be of helpful applications for structures or secondary elements in particular situations like the launching bridges.

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