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"Inspection of Euripus cable-stayed bridge"

INSPECTION OF EURIPUS CABLE-STAYED BRIDGE

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Summary.

Euripus road bridge, located ~80 km north of Athens, is the first Greek cable-stayed bridge. It consists of two approach viaducts of conventional type with prefabricated prestressed beams of length 143.50 and 156.00 m respectively and one central cable-stayed section of three spans 90 –215–90 m, constructed by the cantilever method (cast in situ concrete). The approach viaducts rest by elastomeric bearings on twin frame piers, founded on the rocky subsoil of the area; on the contrary, at the central bridge, the cable-stayed deck is monolithically connected to the twin frame pylons, which are founded on piles. The bridge was constructed between 1987 and 1993. Its design included important pioneer elements for that period, the boldest of which was the design of the deck as a solid slab, of a constant thickness 0.45 m, hanged at its edges per 5.90 m, prestressed generally in the transverse direction and locally in the longitudinal one. On the other hand, the cables were old-type, with parallel strands within HDPE pipes, protected with cement grout. From 1993 till now the bridge is under continuous and heavy traffic, exposed to the significant winds and the sea environment of the area; furthermore, it has suffered from some low to medium earthquakes. Unfortunately, during this period, the bridge hasn't been inspected and maintained; the first systematic inspection has been recently performed by the Greek Ministry of Public Works; based on the results of the inspection, the maintenance of the bridge is now being organized. The inspection covered the whole project, namely concrete, reinforcing and structural steel and cable stay system. The results were in general very positive; the extremely thin deck, despite its continuous dynamic loading, as well as the cables and their anchorages didn't present any serious damages. Furthermore, the concrete is in a very good condition, as to the cracks as well as the carbonation and reinforcement corrosion. Some minor damages, detected at the concrete and the cross ties of the cable stay system, are due to construction mistakes. The very good condition has probably been aided by the cement dust from the near cement factory, which covered the concrete and steel elements surfaces, forming a protective film.

1 INTRODUCTION

Euripus Bridge, ~80 km north of Athens, is the first road cable-stayed bridge in Greece; it bridges the continental Greece with Euboea Island, over Euripus channel (fig. 1). It has a total length of ~695 m, of which 395 m constitute the central cable-stayed part and ~300 m the two approach viaducts. The approach viaducts, with 35.875 or 39.00 m spans, have been constructed by using prefabricated prestressed beams, pre-slabs and in situ poured deck slab. The cable-stayed central bridge has three spans 90 – 215 – 90 m, two pylons of height ~90 m and a prestressed solid deck slab of a constant thickness 0.45 m; it is probably the thinnest deck worldwide (longitudinal slenderness l/h = 215 / 0.45 ≈ 475). The cables consist of parallel galvanized strands, within HDPE pipes, filled with cement grout. The bridge was constructed between 1987 and 1993 and given to traffic in June 1993.

The up-to-now behavior of the bridge is evaluated as very good, without obvious external problems, despite the heavy traffic and the frequent local earthquakes. For confirming the aforementioned, the Greek Ministry of Public Works was conducting an extended inspection of the bridge, which covers all the fields and specifically:

- concrete
- reinforcing steel
- stay cables
- structural steel.

The inspection including all the necessary tests was completed by the end of April 2012; its main conclusions will be presented in this paper.

Figure 1: Completed bridge

2 GEOLOGICAL CONDITIONS

The Boeotia formation (continental, left) is formed of Triassic limestone and dolomites, while the Euboea one (island, right) of Cretaceous limestone. In between the two limestone formations lies, with tectonic contacts, an ophiolithic complex of serpentines down to an average depth of 40 m. The upper part of the channel is covered by alluvia and limestone debris of varying depth up to a maximum of 28 m (fig. 2).

Faults of different size occur along the channel with some possibility to be activated during a very strong earthquake. The piers of the approach viaducts belong, for each viaduct, to individual geological blocks.

From a seismic point of view, all the above formations are classified as class A (rocky ground) of EN 1998-1:2004, as it was proven by the onsite cross-hole tests (velocity of shear waves generally higher than 750 m/s and for the endured alluvia between 500 and 730 m/s).

3 DESCRIPTION

3.1 The central bridge

The central bridge consists of 3 spans 90 – 215 – 90 m, seated on the piers M4, M7 and suspended from the pylons M5, M6 by 144 stay cables (fig. 2). The type of support (hold-down devices and elastomeric sliding bearings) at piers M4 and M7 permits free movement and rotation along the longitudinal axis, while transversally the deck is fully connected to the pier. On the contrary, the connection between the deck and the pylons M5 and M6 is monolithic.

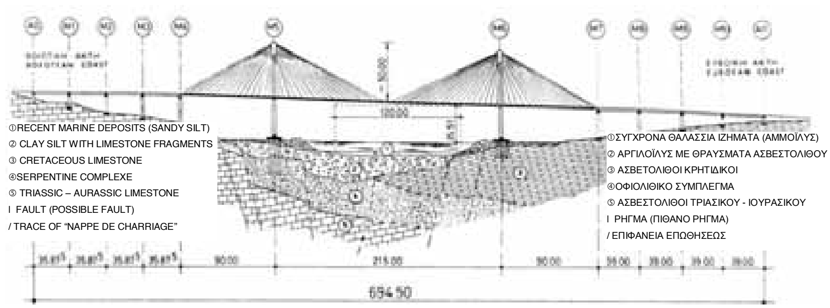

Figure 2: Longitudinal section

The suspension of the deck is realized by two rows of cables, anchored into the deck slab every 5.90 m. The twin pylons M5, M6 are founded at level +1.00. Their heights are 88.35 m for M5 and 84.26 m for M6.

The solid deck slab has a constant thickness 0.45 m everywhere, except for the pylons area where it increases to 0.75 m (fig. 3). It is prestressed transversally by tendons at a spacing of 0.59 m. Longitudinal prestress is realized locally at the central area of the main span, where due to the symmetry, the deck axial force equals zero and at the area of the approach piers, where the horizontal force from the back stay cables is not sufficient to cover the tensile stresses resulting from the high local bending moments.

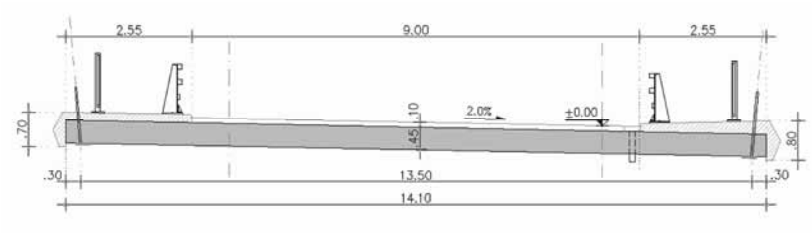

Figure 3: Typical cross-section (central bridge)

The concrete is of a nominal grade B40 (a special class between B35 and B45 according to the old German codes) for the deck slab, the standard reinforcement of grade St 420 and the prestressed reinforcement of grade St 1670/1860. The cables are of VSL type consisting of parallel galvanized strands (7 ÷ 20 ∅6´´, St 1670/1860) within HDPE pipes, protected against corrosion by cement grout.

The minimum breaking load (MBL) of the stays varies between ~1870 and 5350 kN (back stay cables). The maximum stress under the main loads was 0.45 f_u while its fatigue stress range under 40% of the nominal live load was less than 140 N/mm^2.

The pylons have been designed as frames with square hollow columns of variable section 4.00 x 4.00 x 0.50 up to 2.50 x 2.50 x 0.40 (fig. 4).

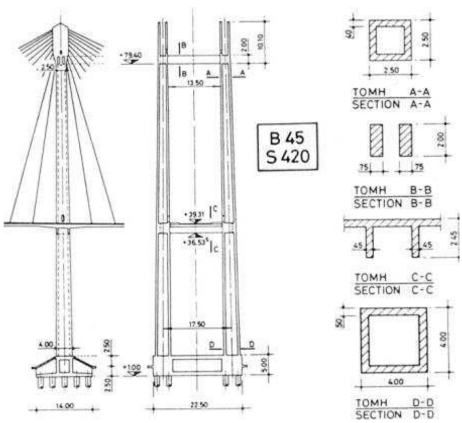

Figure 4: Pylons arrangement and cross-sections

3.2 Approach viaducts

They consist of 4 spans 35.875 m long at the Boeotia (left) coast and 39.00 m long at the Euboea (right) one. They are formed of precast prestressed beams of I cross-section 2.25 m high, 0.20 m web thickness and of an in situ casted prestressed deck slab 0.25 m thick (fig. 5).

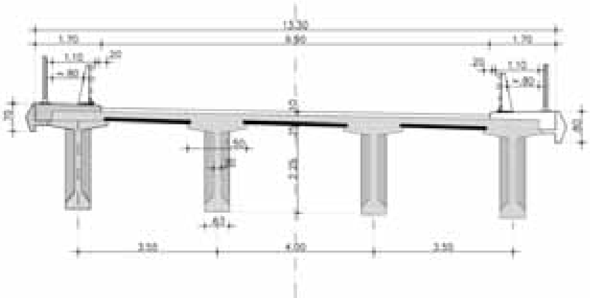

Figure 5: Typical cross-section (viaducts)

The deck slab continues over the joints connecting the individual spans. Cross beams are arranged only over piers in order to simplify the construction, so that the cross sectional load distribution is achieved only through the deck slab.

The main beams were partially prestressed (up to 70%) before placing; the rest of the prestressing was applied after the construction of the deck slab. The latter was transversally prestressed with a force 500 kN/m. Each span is simply supported through elastomeric bearings on the piers. Besides the bearings, additional seismic stoppers are arranged in order to avoid falling of the deck during a very strong earthquake.

The piers were formed as frames with columns of rectangular hollow cross-section, 2.50 m x 2.20 m x 0.30 m and girders of solid cross-section 2.20 m x 2.50 m.

4 INSPECTION PROGRAM

The first systematic inspection of the bridge has been specified by the Greek Ministry of Public Works and was recently executed after ~20 years of continuous operation of the bridge, during which some low to medium earthquakes have been recorded.

The realized program included:

- the geometrical survey of the deck, piers and pylons
- the testing of the concrete and the reinforcing steel
- the testing of the current status of the cables
- the assessment of the cable forces
- the assessment of the actual damping of the cables
- the testing of all structural steel elements
- the checking of the bearings and expansion joints
- the checking of all finishing works (waterproofing, gutters, safety barriers, asphalt layers etc.).

For the inspection of the bottom surface of the central cable-stayed bridge deck, three electric gantries suspended from the deck have been installed. The inspection of the approach viaducts has been realized through movable cranes. The inspection of the cables has been realized through special wheelchairs, suspended from the cable itself and moving along.

5 DECK SURVEY

The survey of the deck and the pylons of the central bridge has been geodetically realized, as well as the survey of the deck and the piers of the approach viaducts.

The survey has been realized during the night, with the bridge closed to traffic, due to the sensitivity presented by this kind of deck to temperature actions and traffic oscillations.

The main results of the survey are as follows:

- Approach viaducts

No difference from the zero measurement (20 years ago) has been recorded, as expected. These viaducts are founded on the rock and have a rigid deck, thus they do not present any significant deformation.

- Deck of central bridge

Due to the very small thickness, the deck is sensitive to temperature actions and especially to the temperature difference between cables and concrete; such a difference is in practice daily developed, due to the large difference of heat capacity between the above structural elements. For example, a temperature difference of 15°C, which can be easily developed in the warm Greek climate in the afternoon of a summer day, leads to a deflection of the deck up to 50 mm.

The survey showed:

- a smooth deflection, without any local discontinuity (which is interpreted as a clear indication of no internal damage), compatible to the theoretical one
- the transverse slope of the deck coincides with the theoretical one, clear indication that both stay cable planes work properly
- the maximum difference between the recorded and the theoretical (after the completion of shrinkage and creeping) deflection is ~60 mm. This very small for such a thin deck difference can be due to differences between the actual and the design data (based on which the construction precamber of the deck has been calculated) or to transient temperature reasons.

6 CONCRETE AND REINFORCEMENT TESTING

The whole bridge is constructed of reinforced and prestressed concrete, except for the cables, their anchorages at the pylon heads / deck and the hold-down devices between the deck of the central bridge and the piers of the approach viaducts.

The testing of the concrete included:

- Visual inspection of all visible concrete surfaces and partial inspection of the foundation surfaces
- Integrity testing of the concrete through electromagnetic waves and ultrasounds in specific critical places of the deck, e.g. at the cable anchorages area
- Testing of the local strength of the concrete in specific critical structural members, e.g. at the pylons base, the pylons head, the deck slab
- Testing of the carbonation depth of the concrete in a significant number of characteristic places

- Testing of the chlorides of the concrete in a significant number of characteristic places of the deck slab
- Measurement of half cell potential in some characteristics places
- Sample checking of the reinforcement cover
- Schmidt hammer testing
- Concrete cores drilling

The visual inspection was conducted by qualified inspectors from the Danish company Ramboll and the Greek one Geotest. During the visual inspection, damages were recorded and photographed individually in order to have a complete photographic record. For the best sorting of the damages, in terms of importance and size, the methodology of the Danish Road Directorate was followed, which classifies damages in the following classes:

- 0: No damage. As new
- 1: Insignificant damage. No action required. Need extra attention during next inspection.
- 2: Minor damage. Repair when convenient. Grouping with major damage.
- 3: Damage. Repair soon.
- 4: Severe damage. Repair is urgent.
- 5: Extreme damage. Action must be taken immediately.

Table 1 presents a first description of the damages and their proposed classification:

S/N	Type of concrete damage	Class
1	Small cracks (0.3 mm)	1 – 2
2	Medium cracks (0.3 ÷ 1.0 mm)	2
3	Large cracks (>1.0 mm)	2 – 3
4	Small surface porous	1
5	Medium surface porous	1 – 2
6	Large surface porous	2 – 3
7	Small delaminations (<∅15)	1 – 2
8	Medium delaminations (<∅30)	2 – 3
9	Large delaminations (>∅30)	3 – 4
10	Visible corroded reinforcement with local delaminations	2 – 3
11	Internal concrete discontinuities (caves)	3
12	Small reinforcement cover	1 – 2
13	Small reinforcement cover with visible corroded rebars	2
14	Small concrete fractures	1 – 2
15	Extensive concrete fractures	3 – 4
16	Salt appearance	1 – 2
17	Moisture appearance	1 – 2
18	Corrosion appearance	2
19	Failures of old surface concrete repairs	2
20	Structural elements with visible deflection	3 – 4
21	Damages due to the drainage system	3
22	Damages (of any type) referring to the users' safety	4 – 5

Table 1: Description and proposed classification of damages

The main results of the inspection and testing are as follows:

- In general the concrete, after twenty years of continuous daily strain and environmental exposure is in a very good condition; the problems which were found out are mostly due to some initial construction defects. The adjacent cement factory has certainly contributed to the very good status of the concrete and reinforcement; the floating cement dust soaked with atmosphere water formed a protective film which has sealed the surface pores of the concrete and has considerably reduced the depth of carbonation.
- The very flexible thin deck slab of the central cable-stayed bridge has behaved extremely well; no significant cracks were detected at any point of the visible bottom surface, even at the area of the cable anchorages, where the introduction of the load is practically concentrated.
- Surface concrete damages have been systematically presented at the points of the gutters, where due to the bad construction details, the rain has been strongly licking the concrete. This damage has been more intense at the approach viaducts due to the complex geometry of the cross-section and smoother at the central bridge, where due to the simple geometry the discharge of the water was easier.
- The pre-slabs of the deck of the approach viaducts have presented unexpected damages at a large extent, namely longitudinal and transverse cracks, crumbles of concrete at the pre-slabs joints and locally large deflections.
- No important damages have been detected at the piers except for some usual surface defects.
- The reinforcement, wherever checked, hasn't presented any corrosion signs or tendency to corrosion.

Table 2 shows the number of records per structural element and the danger index of the damages. In brief, the following damages have been detected:

At the approach viaducts

- 10 damages of class 1
- 37 damages of class 2
- 28 damages of class 3
- 11 damages of class 4

At the main bridge

- 0 damages of class 1
- 14 damages of class 2
- 8 damages of class 3
- 1 damage of class 4

Table 3 shows the concrete strength, the maximum and average carbonation depth, the maximum chloride concentration and the minimum and average reinforcement cover. In brief, the following have been detected:

At the approach viaducts

- Concrete strength f_c>40 MPa
- Max. carbonation depth t_{max}<20 mm
  Mean carbonation depth t_m<10 mm
- Max. chloride concentration <0.05%
- Min. concrete cover t_{min}≈15 mm
  Mean concrete cover t_m≈35 mm

At the main bridge

- Concrete strength f_c > 60 MPa
- Max. carbonation depth t_{max}<20 mm
  Mean carbonation depth t_m≈0.10mm
- Max. chloride concentration < 0.05 %
- Min. concrete cover t_{min}≈15 mm
  Mean concrete cover t_m ≈ 40 mm

	Element	Total number of elements	Number of elements per condition class					
			Class 0	Class 1	Class 2	Class 3	Class 4	Class 5
Boeotian approach viaduct	Abutment	1			1			
	Piers	4				1	3	
	Beams	19		4	11	3	1	
	Cross-beams	8		2	6			
	Soffit plates (per opening between beams)	15			2	10	3	
	Total	**47**		**6**	**20**	**14**	**7**	**0**
Central bridge	Pylons	4			2	1	1	
	Girders	8			6	2		
	Continuance plates	4			2	2		
	Deck plates	3				3		
	Stays areas	3			3			
	Plates between girders	1			1			
	Total	**23**		**0**	**14**	**8**	**1**	**0**
Euboea approach viaduct	Abutment	1				1		
	Piers	4			3		1	
	Beams	16		1	10	5		
	Cross-beams	8	2	3	2		1	
	Soffit plates (per opening between beams)	12			2	8	2	
	Total	**41**	**2**	**4**	**17**	**14**	**4**	**0**

Table 2 : Number of damages per structural element and danger index

Bridge section →	①	②	③
Concrete strength (MPa)	47 ÷ 64	~60	41 ÷ 62
Carbonation maximum depth (mm)	16	~18	~16
Mean value of carbonation depth (mm)	~10	~11	~9
Maximum chloride concentration	0.037%	<0.05%	0.045%
Minimum reinforcement cover (mm)		~16	
Mean reinforcement cover (mm)	~35	~40	~34

Table 3 : Brief results of non-destructive checks (①: Boeotian approach viaduct, ②: Central bridge, ③: Euboea approach viaduct)

7 INSPECTION OF CABLES

The cables were old-type, namely parallel black strands of diameter 0.6´´, St 1670/1860 with an external lead painting, within hard polyethylene pipes, protected by cement grouting (see fig. 6). Externally the polyethylene pipe is protected by a special tape against ultraviolet radiation (PVF tape). The number of the strands varies from 7 to 20 depending on the location of the cable. The deck suspension is realized at every 5.90 m.

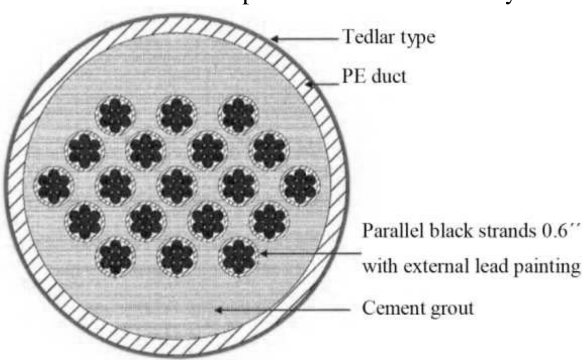

Figure 6: Typical cable cross-section

The inspection of the cables (totally 144) included:

- the external visual inspection of all the cables
- the bottom anchorage internal visual inspection of 60 cables (fig. 7)
- the bottom anchorage internal ultrasound inspection of 60 cables
- the free length internal inspection of 60 cables (fig. 8)
- the upper anchorage internal visual inspection of 60 cables
- the cable force assessment of 60 cables
- the damping factor assessment of 6 cables.

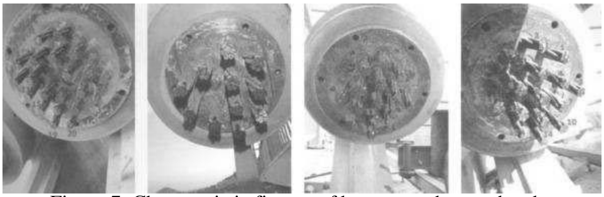

Figure 7: Characteristic figures of bottom anchorage heads

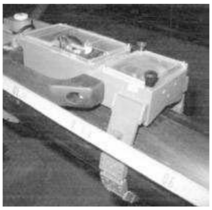

Figure 8: Free length internal inspection by using a special sensor (capacitive gauge inspection)

The external visual inspection of the cables aims at the detection of strands or wire break signs (e.g. longitudinal cracks on the pipe, out-of-roundness defect, outgrowth on the duct, buckling of the cable), the checking of the quality of the cement grout (integrity, leakage, white paste) (fig. 9) and the checking of the external protection tape (damages due to wear or aging). The aforementioned work, aided by a light hammer, has been performed by specialized inspectors moving all along the stay cables, from the upper anchorage to the bottom one. The inspection has not revealed major problems. The most important finding of the visual inspection has been some wire breaks of the cross ties at the locations of the nodes with the stay cables, which led to their replacement (see §9). Furthermore, systematic damage has been detected in the exterior painting of the HDPE pipes, at their upper part and for a length of ~3.00m, where no protection tape had been placed initially for technical reasons; moreover, at the 44% of the stays have been detected damages of small length (up to 50 cm) at the protection tape of the pipes and small mechanical damages, which have been restored in situ.

Figure 9: Bottom anchorage head. The integrity and the good status of the cement grouting are visible.

The area of the bottom anchorage, which is the most sensitive part of the cable, has been visually inspected and checked further through ultrasounds. The checking has been realized in 60 of the 144 cables, without revealing any problems of corrosion or break of the wires, only some traces of surface corrosion at the heads of the strands, which have been assessed as unimportant; at several anchorages moisture has been traced at the bottom side of the waterproofing joint.

The upper anchorages are considered as the most protected part of the cable; therefore, they have been checked only as to their integrity and quality of cement grout, without any revealed problems.

The cement grout of the pipes, wherever checked, was in a very good condition.

The assessment of the stays forces has been realized through the method of vibrating strings at 60 (of 144) stays. The stays have been manually vibrated and the basic eigenfrequency has been measured by an accelerometer. The force of the stay has been calculated by the formula

$$T = 4 \cdot f_o^2 \cdot L^2 \cdot \mu \tag{1}$$

where

f_o: the basic eigenfrequency (Hz)

L: the active length of the stay (m)

μ: the linear mass (kg/m)

Due to traffic problems, measurements were performed under the presence of vehicles, estimated at 10 kN/m of deck.

All measurements were considered as satisfactory. The measured forces came up by average in 31% of the guaranteed failure force of the stays (F_{guts}) with a standard deviation equal to ~5%. As an exception to the rule, the stays on either side of the pylons presented a force around 16% of F_{guts}.

The right stays systematically show increased forces in comparison to the left ones (in average 1.6%) with a maximum deviation in stay 1a (back stay) of ~150 kN, namely ~10%. The difference is due to the overstressing of this specific stay, already known from the construction period, which had also led to a horizontal displacement of the head of the pylon M5R per ~20 mm towards pier M4.

Tables 4 and 5 present the forces of the stays and their distribution in the left and right plane.

Simultaneously to this force and through the same technique, the effective viscous damping of 6 stays has been measured (including the tubes and the grout) and has been found ranging between 1.0 and 2.5%.

Designation Right Side	Nb of strand	Length between anchorage (m)	Linear mass (kg/m)	Theoretical vibrating length (m)	Average F_0 measured (Hz)	Estimated tension F (kN)	Estimated tension by strand (kN)	F/F_{guts} (%)
H01a	20	100.062	44.69 kg	97.962	0.997	1706.0	85.30	32.13%
H01c	19	96.954	43.51 kg	94.854	0.997	1556.6	81.93	30.86%
H01d	19	96.952	43.51 kg	94.852	0.977	1493.2	78.59	29.60%
H04	15	82.977	33.75 kg	80.877	1.24	1356.5	90.43	34.06%
H07	13	68.796	31.39 kg	66.696	1.358	1029.3	79.18	29.82%
H10	11	55.861	29.03 kg	54.061	1.612	881.6	80.14	30.19%
H13	9	46.462	21.74 kg	44.662	2.175	820.4	91.15	34.33%
PYLON M5								
H16	7	43.769	19.39 kg	41.969	1.599	349.2	49.88	18.79%
H18	9	47.078	21.74 kg	45.278	2.256	907.5	100.84	37.98%
H21	11	56.905	29.03 kg	55.105	1.676	990.4	90.03	33.91%
H24	13	70.109	31.39 kg	68.009	1.342	1045.9	80.45	30.30%
H27	15	84.452	33.75 kg	82.352	1.249	1426.9	95.13	35.83%
H30	17	100.046	36.11 kg	97.946	1.062	1561.1	91.83	34.59%
H32	17	109.593	36.11 kg	107.493	0.972	1575.0	92.65	34.90%
MID SPAN								
H35	17	107.945	36.11 kg	105.845	0.965	1506.1	88.59	33.37%
H37	17	98.448	36.11 kg	96.348	1.076	1550.7	91.22	34.36%
H40	15	82.98	33.75 kg	80.88	1.258	1397.0	93.13	35.08%
H43	13	68.796	31.39 kg	66.696	1.42	1125.6	86.59	32.61%
H46	11	55.861	29.03 kg	54.061	1.646	919.5	83.59	31.48%
H49	9	46.461	21.74 kg	44.661	2.162	810.8	90.08	33.93%
H51	7	43.524	19.39 kg	41.724	1.508	306.8	43.82	16.51%
PYLON M6								
H52	7	43.769	19.39 kg	41.969	1.46	291.0	41.57	15.66%
H54	9	47.078	21.74 kg	45.278	2.099	785.3	87.26	32.86%
H57	11	56.905	29.03 kg	54.805	1.633	929.9	84.53	31.84%
H60	13	70.109	31.39 kg	68.009	1.357	1069.6	82.28	30.99%
H63	15	84.452	33.75 kg	82.352	1.225	1372.6	91.51	34.47%
H66a	19	98.514	43.51 kg	96.414	0.985	1569.7	82.62	31.12%
H66b	19	98.516	43.51 kg	96.416	1.001	1620.5	85.29	32.12%
H66c	20	101.699	44.69 kg	99.599	0.991	1739.8	86.99	32.76%

Table 4: Forces of stays at the right cable stay plane (mean stressing 31.26% of F_{guts} with standard deviation 5.32%)

Designation Right Side	Nb of strand	Length between anchorage (m)	Linear mass (kg/m)	Theoretical vibrating length (m)	Average F_0 measured (Hz)	Estimated tension F (kN)	Estimated tension by strand (kN)	F/F_{guts} (%)
H01a	20	100.295	44.69 kg	98.195	0.952	1561.6	78.08	29.41%
H01c	19	97.187	43.51 kg	95.087	0.975	1494.4	78.65	29.62%
H01d	19	97.185	43.51 kg	95.085	0.981	1513.7	79.67	30.01%
H04	15	83.177	33.75 kg	81.077	1.178	1230.5	82.03	30.90%
H07	13	68.963	31.39 kg	66.863	1.359	1036.6	79.74	30.03%
H10	11	56.000	29.03 kg	54.2	1.602	875.1	79.56	29.97%
H13	9	46.579	21.74 kg	44.779	2.123	786.1	87.34	32.90%
H15	7	43.635	19.39 kg	41.835	1.497	304.0	43.42	16.35%
PYLON M5								
H18	9	47.197	21.74 kg	45.397	2.223	885.9	98.43	37.07%
H21	11	57.047	29.03 kg	55.247	1.609	917.5	83.41	31.42%
H24	13	70.279	31.39 kg	68.179	1.332	1034.8	79.60	29.98%
H27	15	84.655	33.75 kg	82.555	1.217	1362.1	90.81	34.20%

H30	17	100.282	36.11 kg	98.182	1.027	1469.2	86.42	32.55%
H32	17	109.668	36.11 kg	107.568	0.96	1539.7	90.57	34.11%
MID SPAN								
H35	17	108.198	36.11 kg	106.098	0.958	1492.3	87.78	33.06%
H37	17	98.631	36.11 kg	96.531	1.066	1527.8	89.87	33.85%
H40	15	83.177	33.75 kg	81.077	1.232	1347.0	89.80	33.82%
H43	13	68.966	31.39 kg	66.866	1.375	1061.2	81.63	30.75%
H46	11	56.000	29.03 kg	54.2	1.612	886.4	80.58	30.35%
H49	9	46.579	21.74 kg	44.779	2.149	805.3	89.47	33.70%
PYLON M6								
H52	7	43.880	19.39 kg	42.08	1.432	281.5	40.21	15.15%
H53	8	45.205	20.56 kg	43.405	2.106	687.2	85.90	32.35%
H54	9	47.167	21.74 kg	45.367	2.037	742.9	82.55	31.09%
H56	10	53.578	22.92 kg	51.778	1.973	956.6	95.66	36.03%
H57	11	57.047	29.03 kg	54.947	1.593	899.5	81.78	30.80%
H59	12	65.521	30.21 kg	63.421	1.413	969.8	80.81	30.44%
H60	13	70.279	31.39 kg	68.179	1.339	1046.3	80.49	30.32%
H63	15	84.655	33.75 kg	82.555	1.202	1329.8	88.65	33.39%
H66a	19	98.750	43.51 kg	96.65	0.949	1464.8	77.10	29.04%
H66b	19	98.552	43.51 kg	96.452	0.952	1506.0	79.26	29.85%
H66c	20	101.939	44.69 kg	99.836	0.969	1672.7	83.64	31.50%

Table 5: Forces of stays at the left cable stay plane (mean stressing 30.77% of F_{guts} with standard deviation 4.48%)

8 TESTING OF STRUCTURAL STEEL ELEMENTS

The bridge has structural steel elements at the following specific points:

- At the deck, as anchorages of the bottom head of the stays and at the same time as supports of the deck
- At M5 and M6 pylon heads, between the two plates of each head, which bear the upper head of the stays
- At the internal side of the concrete discs at the heads M5 and M6. At this side, the design of the project has foreseen steel plates equipped with shear studs. These steel plates
  - bear the horizontal tensile component of the stays
  - transfer through the shear studs the vertical component of the stays to the concrete discs
  - support the horizontal steel beams, which bear the upper head of the stays
  - constitute an internal formwork for the pouring of the concrete discs.
- Between the piers M4, M7 and the deck. At these points, double pendulum steel bars (hold-down devices) transfer the compressive as well as the tensile load of the deck, allowing at the same time its free movement and rotation.
- At all the aforementioned steel elements the following have been performed:
  - Complete visual check (standard EN 970/2007)
  - Check of the galvanization thickness (standard ISO 19840)
  - Checking of weldings through ultrasound (standards EN 1714/1998 and EN 10160/1997)
  - Checking of weldings through magnetic particles (standard EN 1290 / 1998)

Furthermore, the integrity of the steel beams supporting the upper heads of the cables has been checked through ultrasound (mainly against internal defects).

All the above steel elements have been found in a very good condition, both in terms of surface oxidation and defects in the weldings (cracks, breaks) as well as the webs of the steel beams. The detected errors, mainly in the roots of the weldings, were within the permitted normative limits.

The thickness of the remaining galvanization at all points exceeded 150 μm and in many points 300 μm.

9 INSPECTION OF CROSS TIES

In order to limit the irregular oscillations of the stay cables, after the completion of the bridge and following a careful monitoring of their irregular behavior, two cross ties have been implemented in each cable plane. The cross ties consisted of 2 x 2 strands ∅16 and ∅20, slightly prestressed so that they were permanently under tension. At each node, they were connected to the stays through a special device, whose characteristic element was a brass ring with rounded lips (fig. 10).

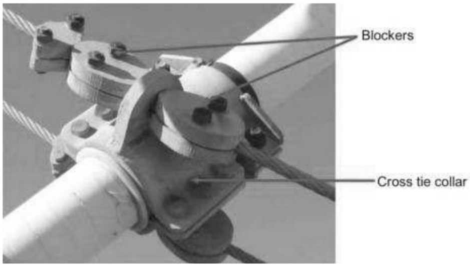

Figure 10: Special device at the nodes between cross ties and stay cables

Due to the geometrical arrangement of the stay cables and the cross ties, an out-of-straightness of the latter presented at each node, which has caused a transverse local force of small size but able to press the strands on the rings; this pressure, systematically increased by the action of the transverse winds, led to the break of some wires of the strands.

Although the problem did not pose any danger for a total strand failure, the removal of the cross ties and the replacement of the strands have been considered as prudent; the detail of the node has been kept as it was, only with an essential improvement of the curvature of the lips of the rings.

10 OTHER BRIDGE ELEMENTS

- Elastomeric bearings

No damages have been detected at the bearings; some construction defects have been only traced at their subbase.

- Expansion joints

All four joints of the bridge have presented damages and thus they have been replaced.

- Gutters

They were from the construction time period problematic and therefore their replacement has been proposed.

- Deck waterproofing

A damage at the waterproofing seems to be probable at a few points of the bridge. For precautionary reasons, its re-construction during the next maintenance has been proposed.

- Lightning protection system

It presented some common age damages, which have been immediately rehabilitated.

11 CONCLUSIONS

The whole bridge, after 20 years of continuous operation and exposure to the sea environment, has not presented any major damages.

The behavior of the central bridge deck has been proven outstanding; this deck, proposed by Prof. J. Schlaich and accepted by the Checker M. Virlogeux, has been considered at the time as pioneer regarding the operational behavior as well as the structural one.

Some minor problems, which have been detected, as the break of wires at the cross ties and surface concrete damages close to the gutters, are mainly due to initial construction problems.

12 CONTRIBUTORS

Project Manager	Greek Ministry of P. W. – Department of Motorway Maintenance
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General Inspector	AKTOR S.A., Athens
Special Inspectors	FREYSSINET INT. & CIE and VINCI CONC.S.A. (stay cables), GEOTEST SA (concrete checking), TOPOMETRIA LTD (deck survey), ATOMDYNAMIC LTD (structural steel checking)

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[4] ITSAK – *Internal Report on the Mantoudi Earthquake 14.10.2008 and the Response of Euripos Bridge* – 2008

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Appendix A: Characteristic damages

Figure A1: Abutment A0 – General view and vertical crack at the whole height

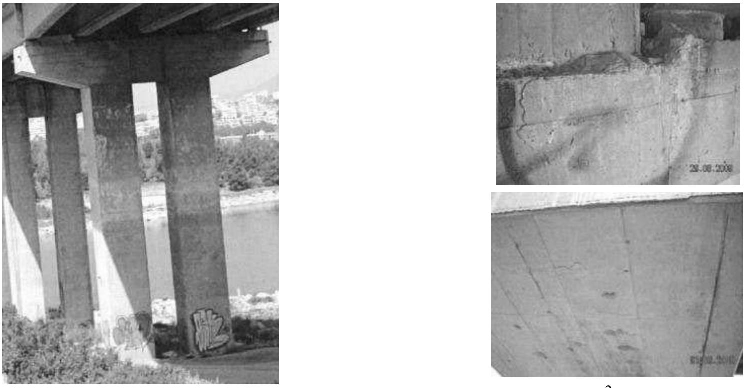

Figure A2: Typical pier of the viaducts, delaminations at an area ~50 · 50cm^2 under the beam bearing at pier M1 and delaminations and visible reinforcement at the bottom part of the girder of pier M3

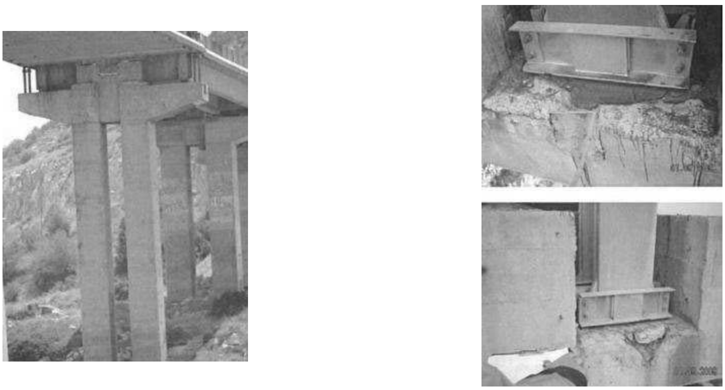

Figure A3: Pier M4 – General view and concrete damages at the base of the hold-down devices

Figure A4 : Pylon M5 – General view and concrete damages e.g. delamination with corroded reinforcement at the column, cracks at previous concrete repairs, delamination at the area of the manhole of the pylon, visible corroded reinforcement at the base of the pylon

Figure A5 : Typical deck of the approach viaducts and visible corroded reinforcement at the bottom flange of the deck beams, concrete corrosion at the area adjacent to gutters and , porous at the bottom flange of the prefabricated beam

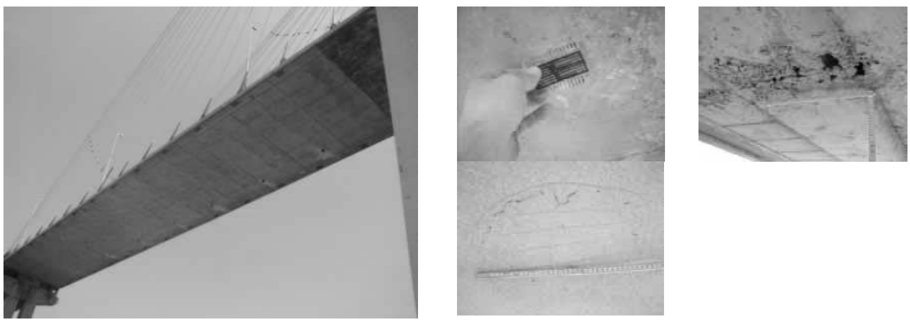

Figure A6 : Deck slab of central bridge, very small cracks of ~0.10 mm width at the bottom surface of the slab, locally large porous and delamination at the bottom surface of the slab

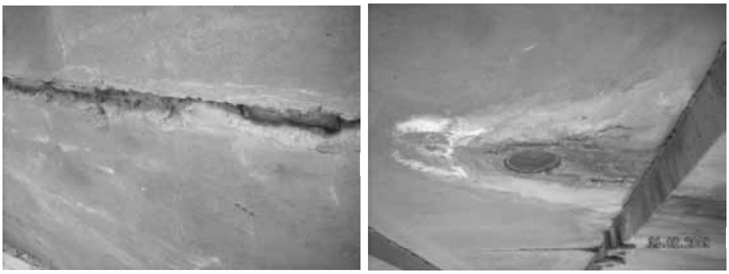

Figure A7 : Damages at the joint between two pre-slabs and moisture at the area adjacent to gutter

Figure A8: Very small crack and visible deflection at the pre-slabs

Figure A9: Fracture of pre-slab edges and visible pre-slab reinforcement, failed repair

Figure A10 : Waving of the cable pipe (not the strands) due probably to loose hanging during construction, minor damage of the protection tape; the 44% of the stays presented such small damages of length up to 50cm

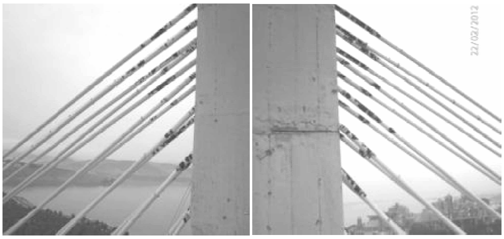

Figure A11 : Damages of the protection painting of the stay cables near the pylons; at this area no special protection tape has been applied (due to the inability of the wrapping machine) and it has been replaced by painting, which has been destroyed over the time.

Figure A12 : Damages at two stays, exposed to a small fire; the protection tape has totally been destroyed and the polyethylene pipe has been damaged at its surface.

(a) (b) © (d)

Figure A13 : Stays bottom head (a) after the opening of the head, (b) after the cautious removal of the cement grout, (c) re-position of the cover with the special inlet of wax injection and (d) detail of strand and wedges

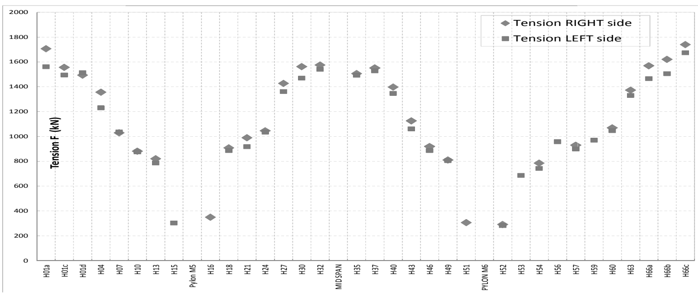

Figure A14 : Comparison of stays tension between the left and right cable plane

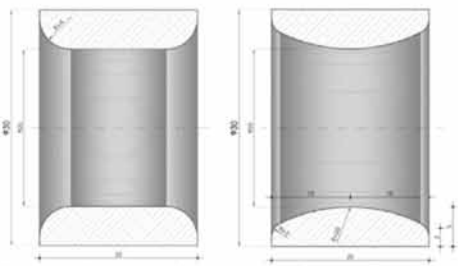

Figure A15 : Brazen ring deviator. Old and new layout.

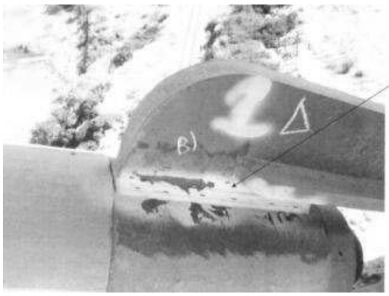

Figure A16: Anchorage of typical stay. Check of anchor head welding.

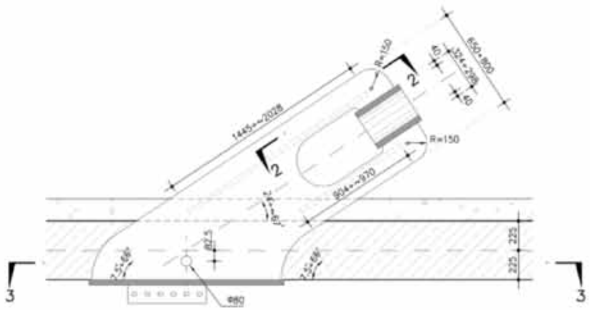

Figure A17: Longitudinal section of typical stay anchorage