THE EVALUATION OF STRESS-STRAIN STATE OF ASPHALT BRIDGE JOINTS FOR HIGH STRUCTURAL DURABILITY

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ABSTRACT

Bridges joints are designed to accommodate structure movement, such as to ensure the structural functionality which could be engaged by traffic loads and thermal and seismic stresses, maintaining the road surface continuity.

Buried joints are widely used in viaducts characterized by an isostatic structure. They represent a suitable solution for the management of these infrastructures because of their inexpensiveness and ease of installation. Since their performance depends on mechanical behaviour of the asphalt concrete placed as filling in of the area around the joint, an evaluation of stress-strain state affecting the asphalt concrete during the service life is needed.

On the basis of criteria of construction science, the Authors introduce a methodology of calculation, which allow to predict with excellent approximation the stress-strain state affecting the above-mentioned joints, furnishing a simple but powerful tool for the definition of the performance needed degree of the asphalt concrete making up the joint, in order to avoid a premature failure.

Thermic stresses, seismic forces, traffic loads and braking action were taken into account. A FEM analysis by the implementation of SAP2000 software was performed in order to evaluate the validity and the accuracy of the proposed methodology.

Results show that the introduced calculation methodology allows to attain a greater durability of the structure, obtaining economic advantages due to the reduction of the maintenance works during its service life.

Keywords: buried joints, influence line, FEM model

1. INTRODUCTION

Roads infrastructures development is connected to the construction of great structures, like bridges and tunnels, which allow the traffic outflow, providing comfort and safety. Bridges require the inclusion of joints, which permit all movements due to thermic stress, braking forces, etc., without affecting the functionality or performance of the structure. Depending on the structural scheme and the movement requirements, different types of joints are used. The isostatic scheme consist of a simple supported beam: the maximum span in an multi-span isostatic beam bridge is about 50m between the piers centres (longer spans characterize iperstatic structures). The stresses to which joints are subjected, depend on several factors, like seasonal temperature variations, seismic forces, traffic loads, etc.

Buried joints are widely used in isostatic beam bridges, because of their inexpensiveness and ease of installation. They consist of a membranes system, which is connected to the structure and to the overhanging pavement layer. The filling in of the area around the joint is made using the same bituminous mixture of the overhanging pavement layer, such as to maintain a smooth riding surface across the gap.

In this paper the Authors introduce a methodology of calculation, which allow to predict with excellent approximation the stress-strain state affecting the above-mentioned joints, furnishing an useful tool for the definition of the performance needed degree of the asphalt concrete making up the joint.

2. FORCES AFFECTING THE STRUCTURE

In bridge joint design the knowledge of the stress-strain state affecting the structure is fundamental. Figure 1 shows the movements (concrete shrinkage, structural settlements included) and the load typologies, to which is subjected the bridge during the service life.

Forces can be characterized on the basis of the time and the frequency of application: traffic loads and stresses due to variations of daily temperature present higher value of frequency; on the contrary the stresses due to seasonal temperature variations show low frequency.

The paper present the analysis of the stress-strain state induced by the structure weight, the traffic load, the braking force, the variations of seasonal and daily temperature and the seismic forces.

Referring to construction guidelines [5], bridges are subject to:

- Permanent forces:
  - g_1 weight of the structural elements and of those not included in the structure;
  - g_2 permanent loads;
  - g_3 earth thrust, hydraulic thrust, etc.
- Distorsions:
  - ε_1 distorsions and design under-stressing;
  - ε_2 shrinkage;
  - ε_3 temperature variations;
  - ε_4 viscosity;
  - ε_5 restraining settlement joints.

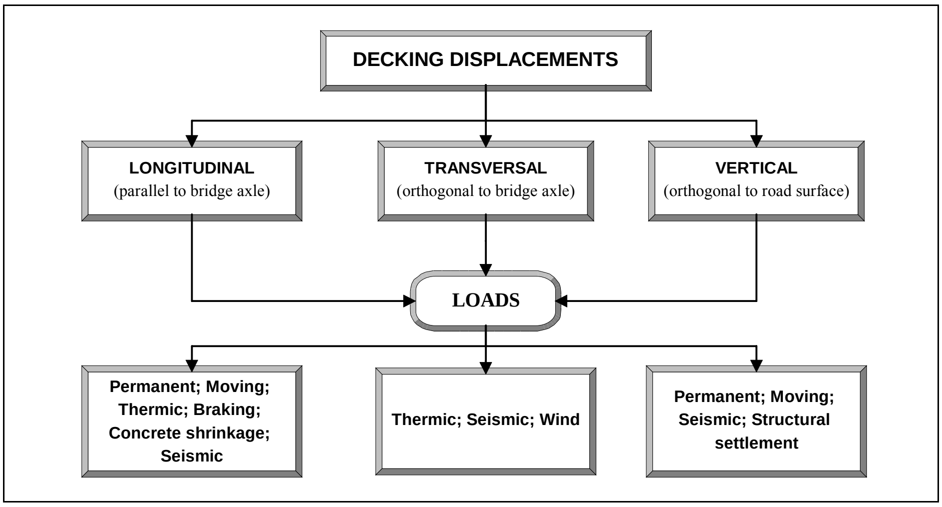

Figure 1 – Displacements and typologies of loads

- Moving loads (due to the traffic):
  - Load scheme 1 –single/uniform loads (Figure 2a);
  - Load scheme 2 - wheels couple of 180kN single axle load (Figure 2b);
  - Load scheme 3 - single axle load of 100kN, having the side of the tyre print equal to 0,30 m;
  - Load scheme 4 - single axle load of 10kN, having the side of the tyre print equal to 0,70 m;
  - Load scheme 5 - load applied with a contact pressure of 4,0kN/m^2;
  - Load scheme 6 a,b - structures with a span over 300m.
- Moving load increment due to dynamic actions:
  The increment is defined as $q_2=(\Phi-1)\ q_1$, where q_1 is related to the most onerous load scheme and the dynamic coefficient Φ assume the following values:
  - Spans with L≤10m — Φ=1,4
  - Spans with 10≤L≤70m — Φ=1,4 - (L-10) / 150
  - Spans with L≥70m — Φ=1,0
- Braking longitudinal forces:
  Braking force q_3 is assumed to act on the surface, along the carriageway axle, with magnitude equal to 1/10 of the most heavy column of load for each carriageway.
- Seismic forces:
  In this case the mass related to the proper weight and to the permanent overloads are taken into account.

3. CALCOLUS OF THE FORCES AFFECTING THE JOINT

The considered structure model is an two-span beam bridge, in which the spans are connected by buried joints. The referring scheme is a simple supported cantilever beam, where L1 is the length between the supports and L is the cantilever length, of 28,0m and

0,80m respectively. The assumed road section consists of a carriageway 12,50m wide and of two insurmountable and unprotected sidewalks, each 1,50m wide; the bearing structure was characterized by four steel Fe 510 double "T" beams, placed at a distance of 2,68m (measured by the axis) and connected by a Rck350 concrete casting. The concrete thickness was assumed equal to 0,30m.

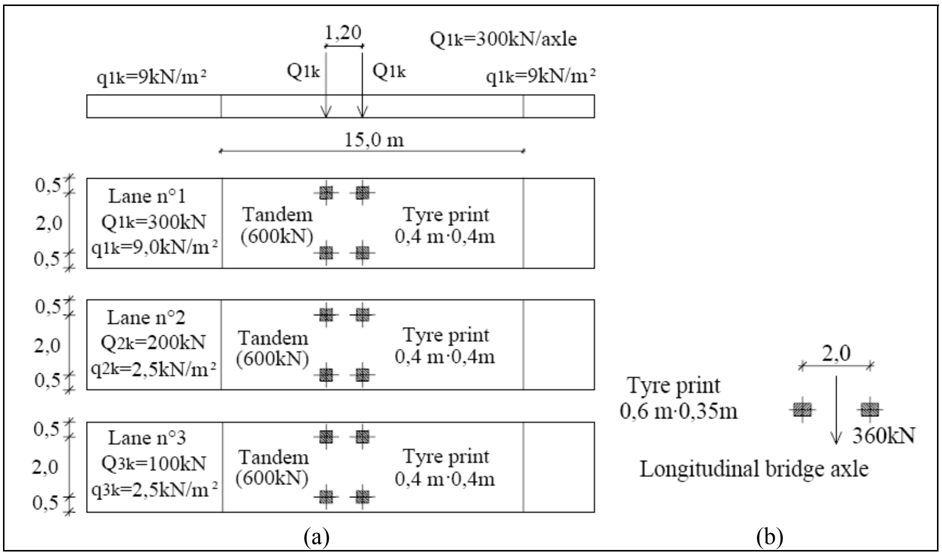

Figure 2 – Load schemes

3.1 Influence lines

Because of dynamic and permanent loads, joints are subjected to stresses due to the beams head movements. The method of the influence lines of rotations per vertical moving load on simple supported beam is usually used (Figure 3). Betti Theorem [1] concerns a plain structure subjected to two load schemes: the one is real and balanced, the other is a dummy scheme of congruent displacements with the assigned restrains. The analytical expression:

$$\sum_i F_i^{(1)} \cdot \delta_i^{(2)} + \sum_j C_j^{(1)} \cdot \Delta_j^{(2)} = \sum_i F_i^{(2)} \cdot \delta_i^{(1)} + \sum_j C_j^{(2)} \cdot \Delta_j^{(1)} \qquad \text{(Eq. 1)}$$

is obtainable applying the virtual works criterion to the above-mentioned schemes.

Taking into account a finite number of loads, assumed $C_1=0$, $\Delta_2=0$, $F_1=F_y$, $\eta_2=\eta$, $C_2=M_2$, $\Delta_1=\phi_i$, $F_2=0$, the expression turns to :

$$\phi_i \cdot M_2 = F_y \cdot \eta \Rightarrow \phi_i = F_y \frac{\eta}{M_2} \qquad \text{(Eq. 2)}$$

Assuming $M_2=Fy=1$, it is possible achieve the equality $\phi_i=\eta$. The evaluation of settlement η is performed resolving the elastic line and by the calculation of the diagram of rotation per vertical moving loads (ϕi^{Fy}). The elastic line equation is easy obtainable applying the Hamilton Criterion[1]. It asserts "*the variation before the Functional T-E$_T$*

during the interval [t_1, t_2] is equal to the work produced by friction forces affecting the system during the same", i.e.:

$$\int_{t_1}^{t_2} \delta(T - E_T)dt = \int_{t_1}^{t_2} \left(\int_S \nu \frac{\partial \overline{u}}{\partial t} \delta u dv + \int_{F_t} \nu_1 \frac{\partial \overline{u}}{\partial t} \delta u ds \right) \cdot dt \qquad \text{(Eq. 3)}$$

where S and F_t are the restrained and free surfaces respectively .Assuming $E=T-E_T$ (Lagrange Functional) and supposing no friction ($\nu=\nu_1=0$), the (Eq.3) turns to:

$$\int_{t_1}^{t_2} \delta L dt = 0 \qquad \text{(Eq. 4)}$$

so the elastic line v(x) is obtainable taking into account the Lagrange Functional. By the hypothesis of the Eulero-Bernoulli beam, the elastic line depends on four constant (A, B, C, D), estimated by applying of bordering conditions:

$$v(x) = \frac{1}{6}Ax^3 + \frac{1}{2}Bx^2 + Cx + D \qquad \text{(Eq. 5)}$$

The scheme related to the influence lines $\Phi_A^{(Fy)}$ e $\Phi_B^{(Fy)}$ are showed in Figure 3 .

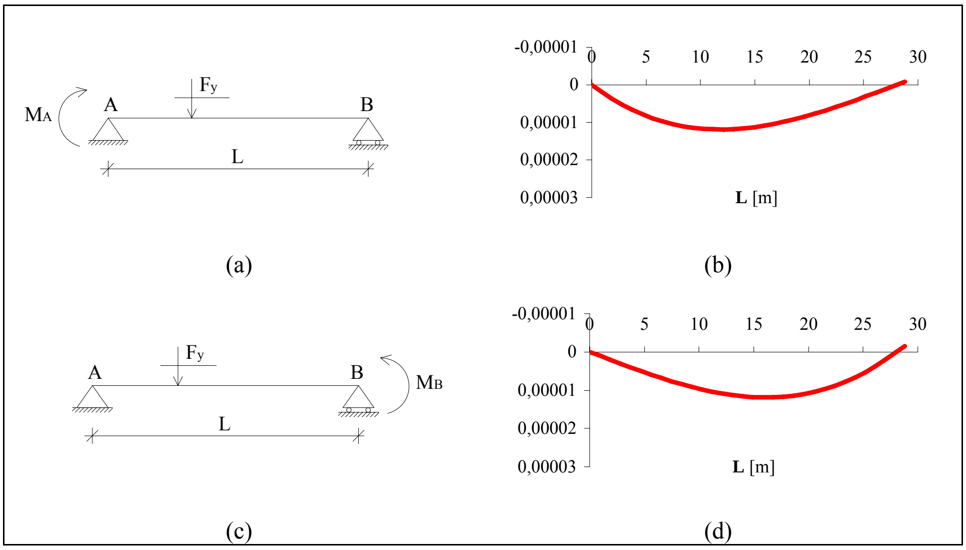

Figure 3 – Schemes and influence lines

As can be seen in Figure 4, because of the permanent load, the point P has a vertical and horizontal displacement, equal to:

$$\begin{cases} u_p = \phi_A \cdot y_n + \phi_{\mathbf{B}} \cdot (H - y_n) \\ v_p = \phi_{\mathbf{B}} \cdot L_1 \end{cases} \qquad \text{(Eq. 6)}$$

where y_n is the ordinate of the neutral axis of the steel-concrete section H high.

The analytical expression of the beams head displacements, carried out on the basis of influence lines and of the Betti Theorem (Figures 4b, 4c and 4d), is the following:

$$u_{P,F} = \frac{F_y}{6EI} \cdot \frac{(x^3 - 3x^2 L + 2xL^2) \cdot y_n + (-x^3 + xL^2) \cdot (H - y_n)}{L}$$
$$v_{P,F} = \frac{F_y \cdot L_1}{6EI} \left(-\frac{x^3}{L} + Lx \right)$$
$$u_{P,q} = \frac{q}{6EI} \cdot \int_a^b \frac{(x^3 - 3x^2 L + 2xL^2) \cdot y_n + (-x^3 + xL^2) \cdot (H - y_n)}{L} \cdot dx \qquad \text{(Eq. 7)}$$
$$v_{P,q} = \frac{q \cdot L_1}{6EI} \cdot \int_a^b \left(-\frac{x^3}{L} + Lx \right) dx$$

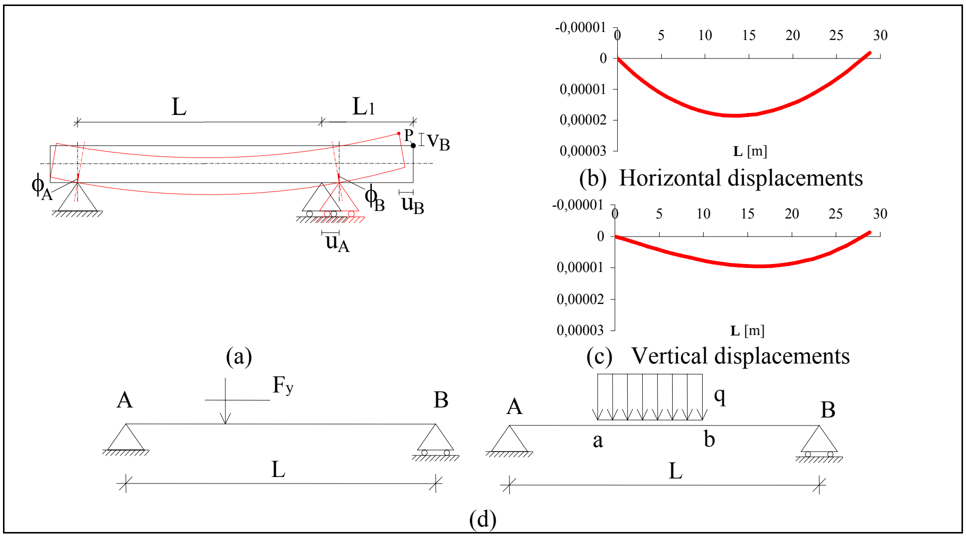

Figure 4 - Displacement due to single and uniform loads

3.2 Permanent loads

On the basis of the distance between two consecutive double “T” beams, an area of 17,36kN/m was assumed as reference for the analysis of permanent loads.

The maximum value of head beam displacements was carried out applying the above-mentioned equations (Eq.7), obtaining 0,6122mm and 0,30611mm for the horizontal and vertical displacement respectively .

3.3 Moving loads

The adopted load scheme is that producing the maximum stress, i.e. the scheme in which there is the highest number of vehicles along the longitudinal axis of the bridge [5]. It can be achieved placing a couple tandem axles load between uniform loads, each of 9kN/m^2 (Figure 5).

By moving of the load on the structure, it was possible verify that the maximum horizontal and vertical displacement occurred in correspondence to a load distribution of 8,80m and 6,00m length respectively. Assuming a span of 28m, the dynamic increment was equal to 28%.

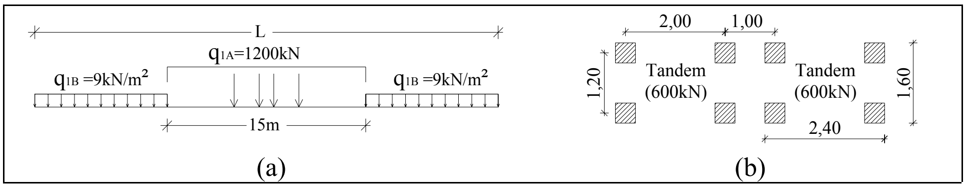

Figure 5 – Moving loads schemes

3.4 Braking force

The braking force of vehicles produces a displacement of the head beam. The motion equation can be carried out by the assumption of the isostatic scheme (Figure 6) with two degree of freedom (Figure 7a), applying the Hamilton criterion (Eq.4).
Since support shear stiffness is greater than joint axial stiffness, one degree of freedom was considered (Figure 7b). Moreover, the force was applied on the structure (Figure 7c), because the maximum stress value in the joint occurs in case of instantaneous stop of the vehicle.

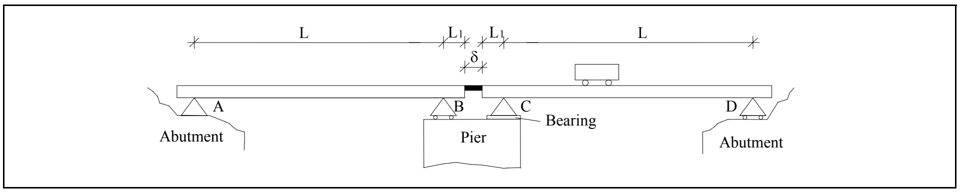

Figure 6 – Isostatic beam scheme

The motion equation is:

$$m \cdot \ddot{x} + c \cdot \dot{x} + k_{app} \cdot x = p(t) \qquad \text{(Eq. 8)}$$

where *m* is the mass of the deck and the vehicles, *c* is the friction, k_{app} the support stiffness and p(t) is a force of trapezoidal type (Figure 7d). Figure 8 shows the displacement variation versus the time and the characteristic parameters (ξ internal dumping, ω pulsation, ω_D dumped circular natural frequency, etc.) of the motion.

3.5 Seismic forces

The analysis of seismic forces was performed assuming the system with one degree of freedom, unforced with a ground displacement (Figure 9).

Once achieved the motion differential equation and defined the initial conditions, it is possible obtain the variation of displacement, speed and acceleration versus the time.

The seismic analysis was performed referring to DM 16/01/1996 [3] and the Guideline 2005 [5]. Supposing a 1st level seismic area, a C type ground and assuming a structure vibration period of 0,0813s, the values of displacements were 1,38mm and 0,16mm applying the DM 16/01/1996 and Guideline 2005 respectively (Figure 10).

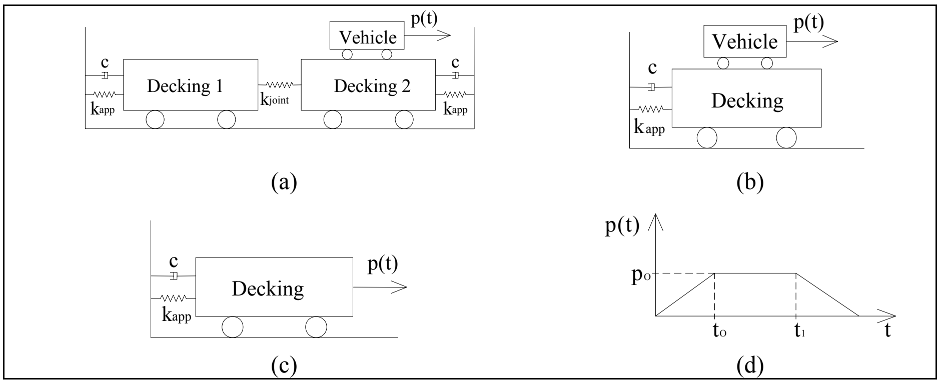

Figure 7 – Dynamic scheme relating to braking forces

k_{app}=1206931,7 kN/m
m=202,1 kNs^2/m
ξ=3,60 %
p(t)=120 kN
x_o=0 m
v_o=0 m/s
ω=77,28 Hz
$ω_D$=77,23 Hz
T=0,081 s
f=12,30 Hz

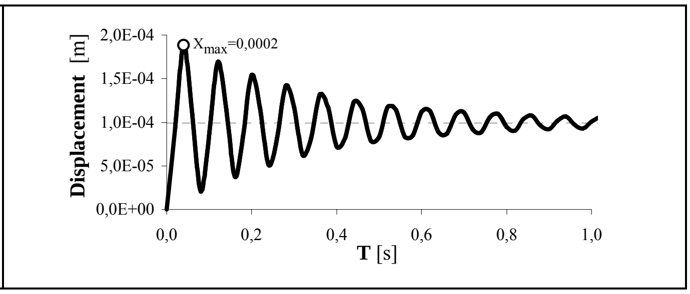

Figure 8 – System with one degree of freedom: motion parameters

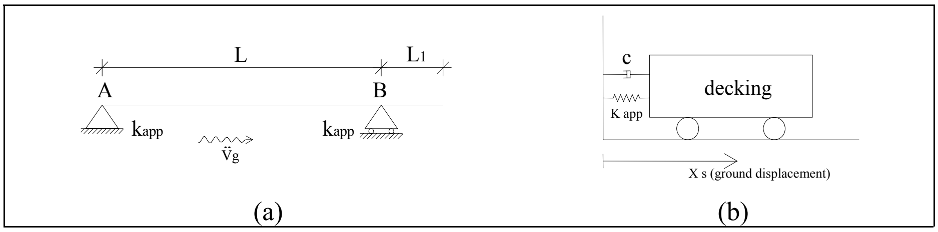

Figure 9 – Dynamic scheme relating to seismic forces

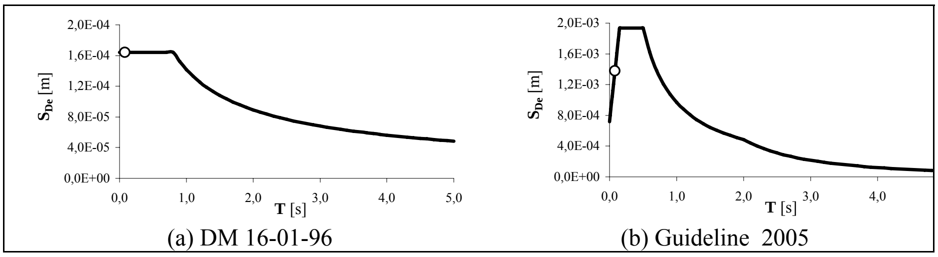

Figure 10 – Response spectrums

3.6 Temperature

In order to represent seasonal and daily temperature variations, two schemes were adopted: one (seasonal) having an uniform distribution; other (daily) with a double triangle distribution, such as to subject to different temperature value the extrados and intrados of the section.

The analysis was conducted choosing temperatures of 10°C, 25°C and -10°C. Table 1 show the results in terms of horizontal U_p and vertical W_p displacement of the head beam, referring to each seasonal (C1, C3 and C5) and daily (C2, C4 and C6) load combination.

Table 1 – Displacements due to the temperature variations

	T = 10±7°C		T = 25±11°C		T = -10±7°C	
	C1	**C2**	**C3**	**C4**	**C5**	**C6**
U_p (mm)	3,974	1,391	9,936	2,186	-3,974	-1,391
W_p (mm)	0,00	0,696	0,00	1,093	0,00	-0,696

4. FEM MODELING

In order to assess the reliability of the results, the Authors performed a FEM analysis by Sap2000®, assuming the same reference scheme (L=28m, L_1=0,80m, δ=0,10m – Figure 6) adopted in the analytic method.

The pavement was modelled with as a two-layer system (wearing-course and binder) with full adhesion between all layers, each 5cm thick. Beam, deck and pavements were discretized using a three dimensional solid element (shell type), adopting a thickening of the mesh in correspondence of the joint.

Since in all analysis performed joints have a strain less than 2%, a linear elastic stress-strain relationship was assumed. The evaluation of Poisson coefficient of the asphalt concrete was performed with reference of AASHTO Guide 2002 [4].

Stresses in the joint are due to elongations, produced by daily temperature variations, and to roto-translations produced by vehicular traffic in the beam extremities. The application time was assumed equal to 1s and 3600s for traffic load and thermic variations respectively. The values of complex modulus and Poisson coefficient are reported in Table 2.

Table 2 - Complex modulus and Poisson coefficient

Parameters	**Critical Winter (T=-10°C)**	**Winter (T=10°C)**	**Summer (T=25°C)**
E^* (MPa)	32840,21	9408,68	2256.24
ν	0,150	0,159	0,357

The connection of joint-deck was modeled with a costraint which allows only horizontal displacements without friction; for the remaining part a constraint which stops all movements was adopted. The load combinations are the follows:

– PMF - full load bridge subjected to the braking force of the vehicles;

– PMF_{stag} - full load bridge subjected to seasonal temperature variations and to braking force of the vehicles;
– PMF_{giorn} - full load bridge subjected to daily temperature variations and to braking force of the vehicles;
– PMS'05 - full load bridge subjected to seismic forces [5].
Numerical results concerning the stress-strain state in the joint are reported in paragraph 5.

5. ANALYSIS OF THE RESULTS

By the results concerning the stress-strain state in the joint area, it can be seen that the difference on the average is about 12%. The values obtained by the FEM analysis show that the introduced analytic method produce a overvaluation of the horizontal displacement Up (between 2,93% and 22,18%) and of the stress state $\sigma_{11(p)}$; and an undervaluation of the vertical displacement Wp (between 5,30% and 28,4%) and of the stress state $\tau_{12(p)}$ (Tables 3 and 4). Results show:

– stress state in the joint decrease as the temperature increases;
– the maximum stress state in the joint is on the left span (Figure 6), in correspondence of the support;

Table 3 – Head beam displacements referring to load combinations

Load combinations		U_p (mm)		W_p (mm)	
		Analytic	SAP2000®	Analytic	SAP2000®
PMF	10°C	3,94	3,70	0,31	0,31
PMT_{stag}		7,73	6,29	0,29	0,33
PMT_{giorn}		5,15	4,69	0,39	0,45
PMS'05		5,13	4,58	0,29	0,32
PMF	25°C	3,94	3,61	0,29	0,34
PMT_{stag}		13,69	12,51	0,29	0,32
PMT_{giorn}		5,94	5,28	0,45	0,48
PMS'05		5,13	4,58	0,29	0,33
PMF	-10°C	3,94	3,61	0,28	0,32
PMT_{stag}		-0,22	-0,20	0,28	0,31
PMT_{giorn}		5,15	4,57	0,38	0,42
PMS'05		5,13	4,58	0,28	0,30

– the maximum stress state is produced by the moving load or by the seasonal temperature variation (critical winter);
– tensile stress is the greatest in correspondence of the joint fibres connected with the pavement;
– assuming the hypothesis of the restrain without friction between joint and deck and of full adhesion between pavement and deck, there is an increment of the stress state between pavement and deck, so it is possible reduce the design stresses;
– the values of joint displacement increase with the temperature increment (the

greater displacement are in the summertime).

Table 4 - Head beam stresses referring to load combinations

Load combinations		$\sigma_{11(p)}$ (MPa)		$\tau_{12\,(p)}$ (MPa)	
		Analytic	SAP2000®	Analytic	SAP2000®
PMF	10°C	1,48	1,37	0,097	0,10
PMT_{stag}		3,21	2,85	0,10	0,12
PMT_{giorn}		1,48	1,37	0,093	0,11
PMS'05		1,50	1,37	0,093	0,11
PMF	25°C	1,09	0,96	0,067	0,071
PMT_{stag}		3,91	3,69	0,12	0,14
PMT_{giorn}		1,07	0,96	0,064	0,071
PMS'05		1,11	0,96	0.065	0,071
PMF	-10°C	6,10	5,25	0,53	0,60
PMT_{stag}		5,11	4,58	0,48	0,54
PMT_{giorn}		5,71	5,25	0,52	0,60
PMS'05		5,96	5,25	0,53	0,60

6. CONCLUSIONS

The paper faces the problem of the determination of the stress-strain state of the underpavement joints of isostatic beam bridges. The study was conducted applying the criteria of the constructions science and processing the data by a numerical implementation, in order to define a simple but effective calculation methodology for the design of above-mentioneted joints. The method reliability was assessed by FEM analyses performed using SAP2000® software. The comparison between the results of the analytical method and those carried out by the FEM model showed that the accuracy level in predicting the stress-strain state on the head beam is about ±12%, showing the potentialities of the method. In conclusion, the paper furnish a simple, inexpensive but powerful tool which allow to define the optimal mix design of bituminous concrete, avoing a premature failure and attaining a greater durability of the structure, with evident economic advantages due to the reduction of the maintenance works during its service life.

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