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# New procedure for bridge analysis of heavy vehicles transit (NTEO)

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ABSTRACT: Infrastructures such as Bridges and Viaducts can be significantly affected by the effects of Heavy Vehicles (H.V.) transit, which represent an unexpected overload, potentially greater than design standard loads. Moreover, Heavy Vehicle transit authorization is a very onerous activity; as a matter of fact, safety check valuations of all the involved structures are required in reasonable time.

NTEO ("*New T.E. Online*") is a new H.V. transit authorization procedure, which aims to provide a semi-automatic method for estimating the assessment of several existing structures in the process of H.V. transit.

The methodology is focused on the general approach of influence lines, related to the most critical sections of the examined structures (e.g. bending moment in the beam mid-span or shear in the beam support).
The traditional definition of influence lines in bridge design is based on the common criteria of transversal distribution of the actions (Courbon, Massonet). Contrastingly, the NTEO procedure uses influence lines extrapolated from an explicit F.E.M. model of each structure involved in the H.V. transit. This choice requires the implementation of many calculation models, but at the same time leads to much more accurate and reliable results than the simplified theories of transversal distribution, especially when these theories can't be applied.

Once models have been created, it is possible to reuse the influence lines for different load scenarios, facilitating authorizations for the following Heavy Vehicle transits.

*Keywords*: highway bridge, heavy vehicle, assessment and evaluation, safety and serviceability

## 1 INTRODUCTION

The Italian highway administration's release of the Heavy Vehicles transit authorization is a complex operation, which requires the assessment of a large number of bridges and viaducts included along the vehicle path, in a time frame compatible with the purpose of the request.

In response to this problem, highway administrations often use simplified authorization procedures, different from an accurate safety assessment of the existing structure in its current state of conservation.

This paper describes the new H.V. transit authorization procedure (NTEO) developed by Speri for Autostrade // per l'Italia. The methodology is intended to provide a semi-automatic appliance for the estimation of structure assessment (in terms of capacity-demand ratio) at the time of H.V. transit, using a general approach based on influence lines.

The influence lines are extrapolated from an explicit F.E.M. model of each structure present in the highway; these lines are related to the most critical sections of the examined structures, and they refer to the H.V. paths potentially most damaging for each safety check (e.g. bending moment in the beam mid-span or shear in the beam support).

The NTEO procedure provides for the implementation of an updatable database of all the infrastructures belonging to the highway network managed by Autostrade // per l'Italia.

This database must contain all the information necessary for the safety checks of the H.V. transit to be authorized: influence lines of each internal force, stresses produced by the permanent loads and strength of each mechanism. The database compilation is a very onerous activity, but subsequently it speeds up the authorization procedure by returning the real-time outcome of H.V. transit on several structures, ensuring safety both in terms of traffic circulation and conservation of the infrastructural estate.

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## 2 METHODOLOGY

### 2.1 *Basic principles*

NTEO procedure is based on the following principles:

- the analyses are carried out in the hypothesis of linear elasticity and superposition;
- each structure is divided into one or more calculation schemes, analyzed using finite element models;
- for each scheme, one or more verification mechanisms are considered. These mechanisms refer only to the main structural elements of the bridge deck (usually the beams), which are affected by the global H.V. transit effects and are conditioning for the safety checks. On the contrary, it is assumed that the mechanisms relating to the other structural elements, such as substructures (supports, piers, foundations) have greater safety margins;
- all the data necessary for carrying out the subsequent verifications, with particular reference to the influence lines, are extrapolated from a finite element model and entered in the database;
- the safety checks are developed in the combination of the ultimate limit state;
- the procedure involves different ways of H. V. transit, in terms of crossing speed and/or other overloads contemporaneity;
- the input data of each highway bridge must be updated and upgradeable, to take into account the progressive increase of knowledge levels (LC) after a campaign of diagnostic tests or the possible formation/evolution of defects found after inspections;
- The H.V. lane is always assumed to be 3.0m wide. This assumption is in favor of safety, given that larger dimensions would reduce the eccentricity of the load.

### 2.2 *Procedure phases*

The procedure is divided into three distinct phases:

- **"Offline" phase**: during which the structural model of the highway bridge is defined and the elements and their sections to be verified are identified, in relation to the structural type, the knowledge level acquired and the "*de facto status*" of the structure.

For each safety check, the demand induced by the permanent loads is evaluated, one or more influence lines are extrapolated by the model, and the integral strength of the specific element is calculated. This phase of the procedure is defined as "offline" because, once processed, it can be archived without requesting other updates;

- **"Online" phase**: The activities included in this phase are surveys, inspections, investigations on materials and construction details, structural monitoring; this information will be continuously updated through a revision of parameters involved in the calculation, such as factorization coefficients, confidence factor and/or strength of the collapse mechanisms considered;

- **"Live" phase**: During this phase, the H. V. transit authorization is produced in real time. This operation requires a few numerical steps based on the data already stored in the previous two phases. The outcome of the safety checks may be negative or positive; in the second case, the Heavy Vehicles must be limited by specific prescriptions or restrictions (for example a travel speed restriction or a contemporary traffic load limitation).

The applicant for the authorization provides the data relating to the H.V. and information on the highway path. Specifically, the following input data will be required:

- vector of the H.V. axle weights;
- vector of the H.V. axle wheelbase;
- $\Delta x_{head}$ is the distance from the H.V. front edge to the first axle;
- $\Delta x_{tail}$ is the distance from the H.V. back edge to the last axle;
- highway entrance tollbooth;
- highway exit tollbooth.

The entrance and exit tollbooths' information uniquely defines the H.V. path direction and therefore the structures crossed. Once the H.V. properties and the concerned highway bridges list are processed, the spreadsheet carries out the safety checks and associates a safety factor to each structure; the safety factor is relative to the most significant mechanism analyzed and is calculated for each of these four transit modes:

1) free transit and free speed (**MT1**);
2) free transit and reduced speed (**MT2**);
3) exclusive transit and free speed (**MT3**);
4) exclusive transit and reduced speed (**MT4**).

Therefore, the applicant for the authorization will receive the final safety check result (positive or negative) and the indications relating to the transit mode.

### 2.3 *Influence lines*

The influence lines represent a useful tool in the calculation of structures, in particular for bridges, since they allow to evaluate the effect induced by a generic load in a given section, when the load itself varies its position on the structure.

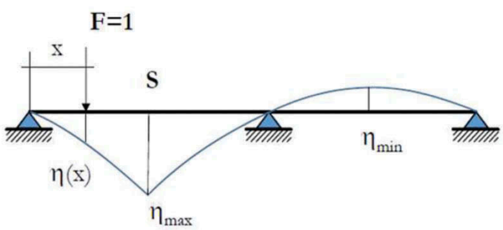


Figure 1. Influence line definition.

In the NTEO procedure context, influence lines are determined through the direct method, which consists of getting these functions by points calculating the generic internal force for different load positions. In most cases, influence lines can be exported directly from the analysis software; therefore, the explicit modeling of all the bridge deck elements (beams, slab) is envisaged.

The F.E.M. model explicitly contains the bridge girder dimension and solves the problem of transversal distribution; for this reason, actually the analysis software generates an influence surface. In accordance with professional practice, the influence surface is considered sufficiently well described by the influence lines of each loading lane, which adopt the usual transverse eccentricities (from the most significant on the deck, to the least relevant in the center of the deck).

Once the model has been created and influence lines have been extrapolated, it is possible to reuse the same lines for different load scenarios to be verified (transit modes), simplifying the H.V. transit authorization release.

### 2.4 *H.V. transit modes*

Permanent loads and traffic overloads generate the internal forces of bridge deck elements considered in the safety checks; these contributions are factored in the ULS combination.

The disposition and the entity of the H. V. simultaneous traffic loads are as follows:

- the H.V. scheme, inside its own lane, is preceded and followed by an indefinite distributed load equal to $9kN/m^2$. This indefinite load can be divided into segments: it is present only in the unfavorable effect positions (for example in multi-span continuous bridges).
- the second loading lane adjacent to the H.V. is occupied by the Load Model no. 1 defined in [2] and it is present only if it produces unfavorable effects;
- the third loading lane distant from the H.V. is occupied by the Load Model no. 2 defined in [2] and it is present only if it produces unfavorable effects.

On each lane (excluding the H.V. lane) and on the remaining area, the bridge deck must be loaded with the frequent values defined in [2]:

$\psi_{1.Q} = 0.75$ is the combination factor for the axle loads;
$\psi_{1.q} = 0.40$ is the combination factor for area loads;

In case of a non-symmetrical influence line or non-symmetrical H.V. scheme, the procedure automatically also considers the H.V. transit in a specular geometric configuration, returning the worst outcome.

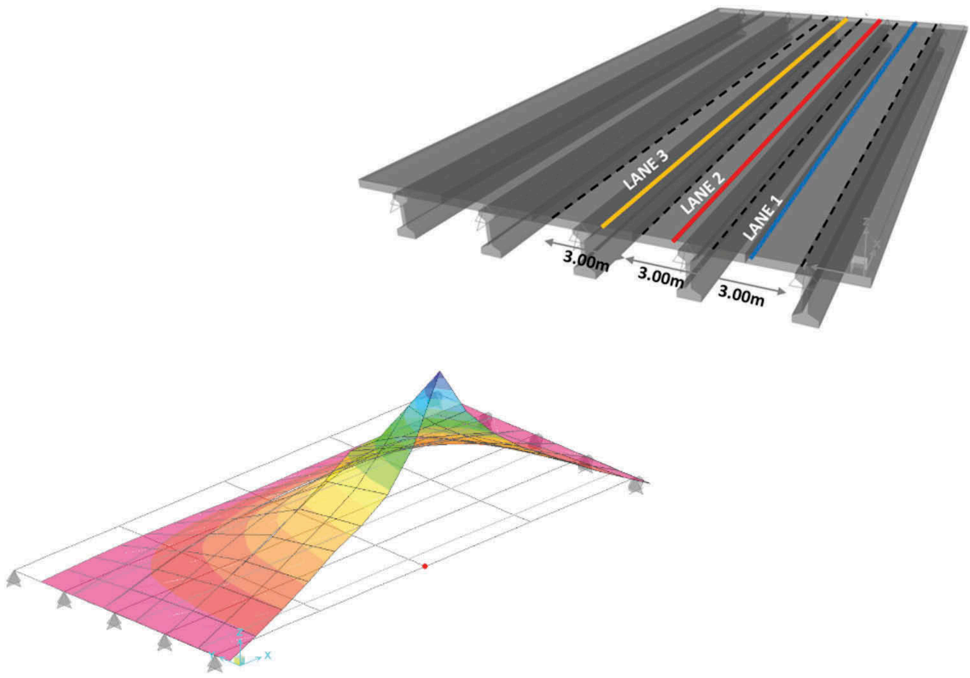


Figure 2. Example of highway bridge girder F.E.M. model and extrapolation of influence surface.

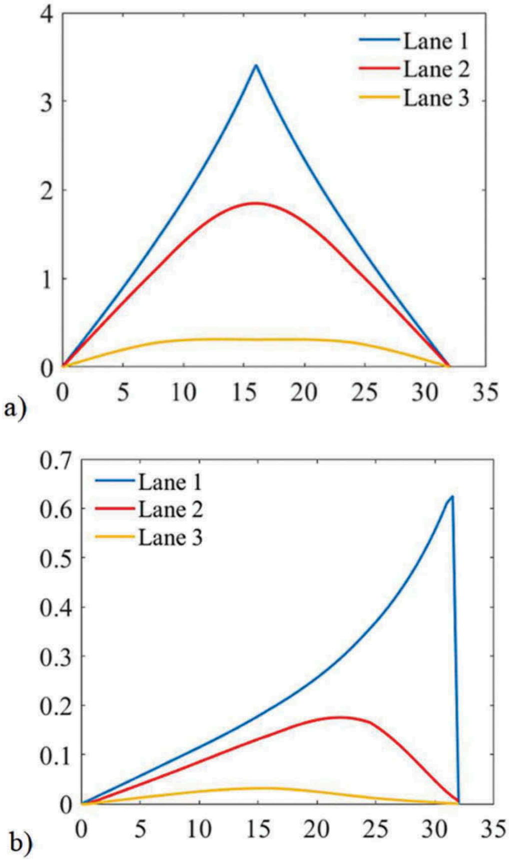


Figure 3. External beam influence lines (Span of 32 m), refer to the 3 load lanes and related to: a) bending moment in the beam mid-span; b) shear in the beam support.

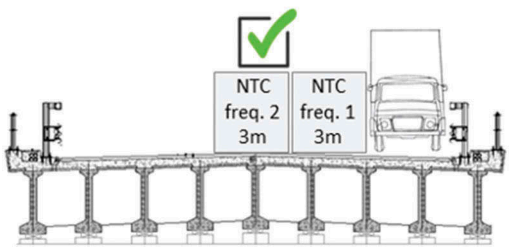


Figure 4. H.V. contemporary traffic load application scheme.

The procedure uses 4 transit modes with decreasing severity:

MT1: free transit (H.V. in any lane and simultaneous presence of general traffic loads) without speed limits;
MT2: free transit (H.V. in any lane and simultaneous presence of general traffic loads) with speed limits;
MT3: exclusive transit (H.V. in any lane without simultaneous presence of general traffic loads) without speed limits;
MT4: exclusive transit (H.V. in any lane without simultaneous presence of general traffic loads) with speed limits;

In case of exclusive carriageway use, there is no simultaneous presence of the general traffic loads but only the Heavy Vehicle as a traffic overload.

According to [3] A.3 (3), the maximum H. V. speed allowed is equal to 70 km/h (MT1 and MT3 conditions). For this transit speed a dynamic amplification coefficient equal to ([3] A.3 (5)) is applied:

$$\phi = 1.4 - L/500 \geq 1$$

On the contrary, for MT2 and MT4 transit modes, a H.V. speed prescription that reduces it by at least half the permitted limit is applied ($v_{max}$ 35 km/h). Since the dynamic effects are generally dependent on the square of the velocity, a reduced dynamic amplification coefficient is conservatively estimated equal to:

$$\phi\_rid = (1 + \phi)/2$$

Differently, Load Models defined in [2] already include dynamic effects; therefore, they should not be reconsidered.

## 2.5 *Safety check format*

The NTEO procedure aims to ensure the safety and structural integrity of highway bridges concerned by H.V. transit. The safety check consists in verifying that the capacity, associated with the generic collapse mechanism, is greater or equal to the total effect produced by permanent and accidental loads.

The safety check outcome is expressed in terms of adequacy coefficient, $\zeta_{TE}$, a safety index, which defines the effect associated with the H.V. In case of structural strength equal to the demand of the collapse mechanism, $\zeta_{TE}$ equals the unit. The expression of the adequacy coefficient is as follows:

$$\zeta_{TE} = \left[\frac{\delta c}{\gamma_C \gamma_M} - \left(\gamma_{g1} D_{g1} + \gamma_{g2} D_{g2} + \alpha_c \gamma_q D_q\right) - \frac{D_{pk}}{\gamma_M}\right] \times \frac{1}{\gamma_{TE} \phi D_{TE}} \quad (1)$$

Where:

- **C** is the structural strength of the section element related to the internal force considered (without defects);

- **δ** is the residual strength factor ($0 \leq \delta \leq 1$), dependent on the defects of the checked section, which takes into account any reduction in capacity;
- $\mathbf{D_{g1}}$ e $\mathbf{D_{g2}}$ represent the effects produced by dead and permanent loads;
- $\mathbf{D_q}$ is the effect produced by the H. V. simultaneous presence of general traffic load on the carriageway;
- $_C$ is a contemporaneity coefficient, assumed equal to 1 in the absence of simultaneous traffic limitations on the H.V. transit (MT1 and MT2) otherwise equal to 0 (MT3 and MT4);
- $\mathbf{D_{pk}}$ is the effect produced by the prestress on the verified mechanism and is generally a beneficial effect, such as in the presence of inclined pre-stressing cables;
- $\mathbf{D_{TE}}$ is the effect produced by the H.V. transit;
- **Φ** is the H.V. dynamic amplification coefficient and can assume different values depending on the transit speed authorized;
- $\boldsymbol{\gamma_{g1}}$, $\boldsymbol{\gamma_{g2}}$, $\boldsymbol{\gamma_q}$ e $\boldsymbol{\gamma_{TE}}$, are the factorization coefficients defined by current legislation [1] [2]; in ordinary conditions these coefficients are assumed, respectively, equal to: $\gamma_{g1}$=1.26, $\gamma_{g2}$=1.26, $\gamma_q$ =1.35 e $\gamma_{TE}$ =1.10.
- $\boldsymbol{\gamma_M}$, named "model coefficient", is an additional safety coefficient introduced by the procedure. It takes into account the modeling uncertainties related to the structural typology (single span deck, slab, multi-span continuous deck, etc.); it considers the level of complexity and reliability of the modeling, amplifying all the effects deriving from the calculation, except for $D_{pk}$;
- $\boldsymbol{\gamma_C}$, named "strength coefficient", is an additional safety coefficient introduced by the procedure that reduces the element strength according to the fragility degree and the danger of the mechanism to be verified.

Both last parameters $\gamma_M$ and $\gamma_C$ are fundamental for the procedure, given that they reduce any approximation errors, only generated by the global model reduction in few lanes.

The definition of $\gamma_M$ coefficients was carried out by associating a "weighting" $M_i$ to all the parameters influencing the modeling; these "weightings" were combined with an additive criterion.

The amplifying factor $\gamma_M$ doesn't replace partial safety factors of the actions envisaged by national technical rules [1] [2], rather it's an additional one. For this reason, it was considered reasonable to assume that the maximum value of $M_i$ is an increase in demand of 5%, from which all the remaining weightings were modulated.

The strength coefficients $\gamma_C$ used in the procedure are shown in the following table:

Table 1. $\gamma_M$ coefficients definition and values of weights $M_i$.

Model coefficient – Additive criterion

$$\gamma_M = 1 + \sum_{i=1}^{6} M_i$$

| | | | |
|---|---|---|---|
| **M1** | Structural typology | concrete beam deck | 0 |
| | | concrete large slab | 0.02 |
| | | concrete box girder | 0.03 |
| **M2** | Section typology | open | 0 |
| | | full | 0 |
| | | cellular | 0.01 |
| **M3** | Material | reinforced concrete | 0 |
| | | c.a.p. | 0.01 |
| | | steel-concrete composite | 0 |
| **M4** | Static scheme | single span | 0 |
| | | continuous | 0.02 |
| **M5** | Bridge deck enlargement | Yes | 0 |
| | | No | 0.03 |
| **M6** | Transfer beams | Yes | 0.05 |
| | | No | 0 |

Table 2. $\gamma_C$ coefficients definition.

| Strength coefficients | | $\gamma_C$ |
|---|---|---|
| **Beam/slab bending moment** | unreinforced section | 1.00 |
| | reinforced section | 1.03 |
| **Beam/slab shear force** | unreinforced section | 1.00 |
| | reinforced section | 1.05 |
| | uncertainty prestress losses | 1.03 |
| | uncertainty cot(θ) [2] | 1.03 |
| **Dapped-End Beams** | | 1.10 |
| **Box girder** | Cast in place bridge | 1.02 |
| | Segmental bridge | 1.04 |

Strength coefficient $\gamma_C$ doesn't replace partial safety factors of materials envisaged by national technical rules [1] [2], rather it's an additional one.

For this reason, it was considered reasonable to assume an increase in demand of 5% as the maximum value of these coefficients (except for Dapped-End Beams, which were assigned an increase of 10%), from which all the remaining coefficients have been modulated.

Currently, strength coefficients introduced in the procedure refer to the following cases:

– presence of existing bending reinforcements on the beams, for example with FRP fibers; their effect

depends on many factors (resin properties, thickness of the composite sheets, delamination phenomenon) which are not always easy to evaluate;
– presence of shear resistant elements of different characteristics (stirrups, bent-up bars, fibers) which can lead to greater uncertainty on the "θ" cotangent evaluation (inclination diagonal cracks);
– strut and tie models on Dapped-End Beams, which require the identification of one or more balanced mechanisms of struts and tie rods inside the element. Strength coefficient takes into account the variability on the strut thickness, the variability on the number of bars considered in each tie rod, and, lastly, the combination criterion used for the mechanisms considered, as there is no clear regulatory reference in this regard;
– executive technology used for box girder bridge decks during the construction phases.

### 2.6 *Estimation of the H.V. severity*

In order to define an evaluation of the H.V. severity, two indices are adopted, the first one dependent on the highway bridge crossed, $IS_S$, the second one linked only to the H.V. conformation (weight, axle distance and number), $IS_{TE}$:

- $IS_S$, severity index related to the historical data, ranging from 1 to 9;
- $IS_{TE}$, severity index related to the Heavy Vehicle, ranging from 1 to 8.

Their maximum index value determines the overall H.V. severity index, IS.

$IS_S$ shows the reference range of the effect (bending moment or shear force) produced by the H. V. transit, $D_{TE}$. The reference ranges are defined through seven fractiles, $Fr_{\_\%}$, evaluated on the effects produced by a H.V. population transited over a period of about a year and a half, and the maximum effect recorded, $max(Historical\ H.V.)$, as shown in Table 3.

These values were determined by neglecting the dynamic amplification effect and the uniform traffic load in the H.V. lane. According to the range in which the H.V. effect is included, the severity index $IS_S$ is determined.

By way of example, a cumulative frequency curve is shown in Figure 5. This curve refers to a single highway bridge and shows the cumulative frequency of a specific effect (in this case shear force) generated by Heavy Vehicles transited in the past (follow-up period). Figure 5 highlights each range corresponding to the $IS_S$ severity levels of the historical data.

Table 3. Reference ranges for severity index related to the historical data, $IS_S$.

| $IS_S$ | Reference ranges |
|---|---|
| 1 | $Fr_{0\%} \leq D_{TE} \leq Fr_{1\%}$ |
| 2 | $Fr_{1\%} < D_{TE} \leq Fr_{5\%}$ |
| 3 | $Fr_{5\%} < D_{TE} \leq Fr_{16\%}$ |
| 4 | $Fr_{16\%} < D_{TE} \leq Fr_{50\%}$ |
| 5 | $Fr_{50\%} < D_{TE} \leq Fr_{84\%}$ |
| 6 | $Fr_{84\%} < D_{TE} \leq Fr_{95\%}$ |
| 7 | $Fr_{95\%} < D_{TE} \leq Fr_{99\%}$ |
| 8 | $Fr_{99\%} < D_{TE} \leq \max(\text{HistoricalH.V.})$ |
| 9 | $D_{TE} > \max(\text{HistoricalH.V.})$ |

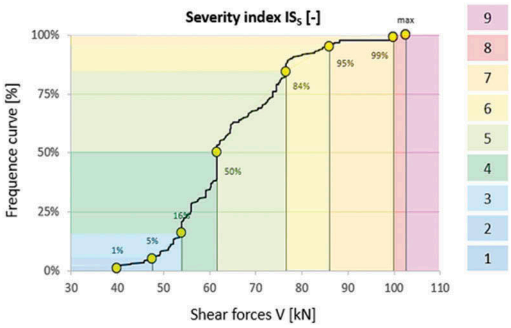


Figure 5. Shear force cumulative frequency curve referred to a highway bridge, generated by real heavy vehicles transited in the follow-up period (a year and a half).

The H.V. severity index, $IS_{TE}$, is determined based on reference ranges associated with different fractiles, similarly to the precedent index. In this case, however, the fractiles refer to a ratio, depending on the Federal Bridge Gross Weight formula (FBGW) [4], evaluated on the whole H. V. population transited in the follow-up period. In this way, the $IS_{TE}$ index only takes into account the Heavy Vehicle properties, such as the axle number, wheelbase and weight.

FBGW is a formula, used by the Department of Transportation of the Federal Highway Administration (FHWA), to establish the maximum weight any set of axles on a motor vehicle may carry on the Interstate highway system. FBGW is used to prevent the transit of vehicles which could damage the infrastructure or cause premature deterioration.

FBGW has the following expression:

$$FBGW = 500\left(\frac{l \cdot n}{n-1} + 12n + 36\right)\ [lb] \qquad (2)$$

Where:

– FBGW is the overall gross weight on any group of two or more consecutive axles to the nearest 500 pounds;
– l is the distance in feet between the outer axles of any group of two or more consecutive axles;
– n is the number of axles in the group under consideration.

Fractiles are evaluated according to the ratio $R_{FBGW}(n, l)$, defined as:

$$R_{FBGW}(n,l) = \frac{W_{TE}(n)}{FBGW_{\max imum}(n,l)} \quad (3)$$

Where:

- $W_{TE}(n)$ is the total weight of the Heavy Vehicle to be authorized;
- $FBGW_{maximum}(n, l)$, is the maximum vehicle load, calculated through (2), depending on H. V. axles number and wheelbase;

Both $W_{TE}(n)$ and $FBGW_{maximum}(n, l)$ are determined depending on "n" and "l" in order to maximize the ratio $R_{FBGW}(n, l)$.

The index is defined by referring to the following ranges:

Table 4. Reference ranges for severity index related to the Heavy Vehicle, $IS_{T.E.}$

| $IS_{TE}$ | Reference ranges |
|---|---|
| 1 | $Fr_{0\%} \le R_{FBGW}(TE) \le Fr_{1\%}$ |
| 2 | $Fr_{1\%} < R_{FBGW}(TE) \le Fr_{5\%}$ |
| 3 | $Fr_{5\%} < R_{FBGW}(TE) \le Fr_{16\%}$ |
| 4 | $Fr_{16\%} < R_{FBGW}(TE) \le Fr_{50\%}$ |
| 5 | $Fr_{50\%} < R_{FBGW}(TE) \le Fr_{84\%}$ |
| 6 | $Fr_{84\%} < R_{FBGW}(TE) \le Fr_{95\%}$ |
| 7 | $Fr_{95\%} < R_{FBGW}(TE) \le Fr_{99\%}$ |
| 8 | $R_{FBGW}(TE) > Fr_{99\%}$ |

By way of example, a cumulative frequency curve is shown in Figure 6. This curve shows the cumulative frequency of the ratio $R_{FBGW}(n, l)$, referred to Heavy Vehicles transited in the past (follow-up period). The figure highlights each range corresponding to the $IS_{T.E}$ severity levels of the historical data.

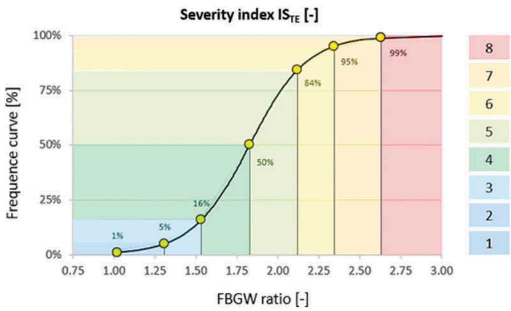


Figure 6. $R_{FBGW}(n, l)$ cumulative frequency curve referred to a set of real heavy vehicles transited in the follow-up period (a year and a half).

The final severity index IS can assume values between 1 and 9, and is equal to the maximum between the two indices previously described:

$$IS = \max(IS_S; IS_{TE}) \quad (4)$$

## 3 CASE STUDY

### 3.1 *Benchmark path examined*

The benchmark path used to test the calculation procedure is an Italian highway. This path choice is due to the almost absence of highway junctions, in order to uniquely identify a set of bridges to be verified.

This path crosses 37 highway bridges, made up of reinforced concrete girders and large slabs, both in c. a.p.; moreover, Dapped-End Beams are present in some of the bridges analyzed.

For the case study, the capacities were calculated starting from the design strength and applying a confidence factor CF=1.35, corresponding to a knowledge level LC1 [2]. About the prestress losses, values predicted by the designer have been increased by 50%.

### 3.2 *H.V. sample used in the simulation*

By way of example, a H.V. transit on the benchmark path has been simulated. The H.V. sample consists of a weight of 110 tons and total length equal to 24.51m, as illustrated in Figure 8:

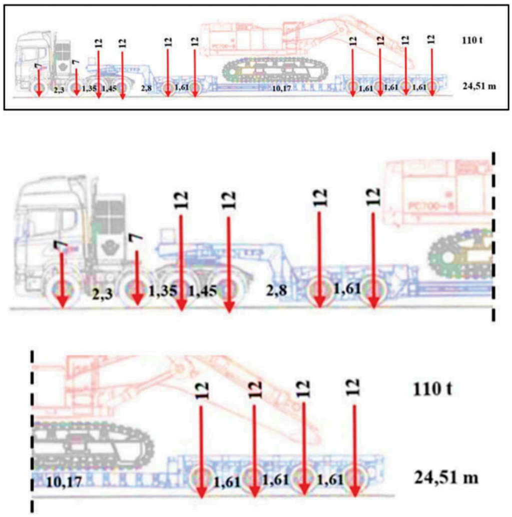


Figure 7. H.V. scheme used in the simulation.

The severity index associated with the H. V. sample is equal to 9 for all the highway bridges belonging to the benchmark path, considering historical data of vehicles carried on the benchmark highway in a follow-up period of a year and a half.

### 3.3 *Safety check outcomes*

The results of safety checks are expressed in terms of adequacy coefficient [1] referring to the H.V.

| $\zeta_{TE}$ |
|---|
| $\geq 1.2$ |
| $1 \leq \bullet < 1.2$ |
| $0.8 \leq \bullet < 1.0$ |
| $< 0.8$ |

| Bridge code | Bridge code | $\zeta_{TE}(MT1)$ | $\zeta_{TE}(MT2)$ | $\zeta_{TE}(MT3)$ | $\zeta_{TE}(MT4)$ |
|---|---|---|---|---|---|
| 1 | 03000000 | 1.60 | 1.69 | 1.71 | 1.82 |
| 2 | 03000300 | 0.90 | 0.95 | 1.09 | 1.17 |
| 3 | 03000600 | 1.13 | 1.17 | 1.37 | 1.43 |
| 4 | 03006100 | 0.85 | 0.90 | 1.01 | 1.09 |
| 5 | 03007300 | 0.94 | 1.00 | 1.10 | 1.19 |
| 6 | 03013500 | 0.94 | 1.00 | 1.10 | 1.19 |
| 7 | 03014800 | 0.95 | 1.02 | 1.11 | 1.20 |
| 8 | 03015100 | 0.94 | 1.00 | 1.10 | 1.19 |
| 9 | 03015400 | 0.91 | 0.96 | 1.03 | 1.09 |
| 10 | 03015800 | 1.18 | 1.24 | 1.38 | 1.46 |
| 11 | 03016500 | 0.96 | 1.03 | 1.11 | 1.20 |
| 12 | 03019800 | 1.59 | 1.70 | 1.77 | 1.91 |
| 13 | 03022600 | 1.29 | 1.38 | 1.43 | 1.55 |
| 14 | 03022800 | 0.61 | 0.64 | 0.71 | 0.75 |
| 15 | 03022900 | 1.08 | 1.12 | 1.29 | 1.34 |
| 16 | 03023200 | 1.03 | 1.10 | 1.23 | 1.32 |
| 17 | 03023300 | 1.05 | 1.10 | 1.22 | 1.30 |
| 18 | 03024000 | 1.11 | 1.19 | 1.32 | 1.43 |
| 19 | 03027300 | 0.96 | 1.01 | 1.07 | 1.14 |
| 20 | 03029700 | 0.81 | 0.87 | 0.88 | 0.95 |
| 21 | 03032200 | 1.08 | 1.14 | 1.21 | 1.27 |
| 22 | 03039400 | 1.04 | 1.11 | 1.15 | 1.24 |
| 23 | 03039900 | 0.87 | 0.93 | 0.94 | 1.02 |
| 24 | 03041100 | 1.07 | 1.12 | 1.20 | 1.27 |
| 25 | 03041800 | 1.09 | 1.16 | 1.22 | 1.31 |
| 26 | 03042200 | 1.04 | 1.11 | 1.17 | 1.26 |
| 27 | 03042600 | 0.76 | 0.81 | 0.81 | 0.87 |
| 28 | 03043900 | 0.98 | 1.06 | 1.06 | 1.16 |
| 29 | 03045800 | 1.06 | 1.12 | 1.12 | 1.22 |
| 30 | 03052500 | 0.94 | 1.01 | 1.05 | 1.14 |
| 31 | 03052800 | 1.01 | 1.08 | 1.12 | 1.18 |
| 32 | 03053200 | 0.80 | 0.86 | 0.88 | 0.94 |
| 33 | 03053300 | 0.96 | 1.04 | 1.05 | 1.14 |
| 34 | 03054100 | 0.87 | 0.91 | 1.00 | 1.05 |
| 35 | 03054300 | 0.87 | 0.92 | 1.00 | 1.06 |
| 36 | 03056400 | 1.20 | 1.31 | 1.29 | 1.42 |
| 37 | 03056600 | 0.77 | 0.84 | 0.81 | 0.88 |

| | $\zeta_{TE}(MT1)$ | $\zeta_{TE}(MT2)$ | $\zeta_{TE}(MT3)$ | $\zeta_{TE}(MT4)$ |
|---|---|---|---|---|
| | V norm. | V rid. | V norm. | V rid. |
| $\geq 1.2$ | 4 | 5 | 15 | 20 |
| $1 \leq \bullet < 1.2$ | 15 | 23 | 20 | 17 |
| $0.8 \leq \bullet < 1.0$ | 20 | 13 | 6 | 4 |
| $< 0.8$ | 3 | 1 | 1 | 1 |

Figure 8. Safety check outcomes, in terms of adequacy coefficient, related to each highway bridge belonging to the benchmark path.

($\zeta_{TE}$). The adequacy coefficient $\zeta_{TE}$ is evaluated for all the verification mechanisms and only the worst is returned.

Safety check outcomes regarding the H.V. sample transit on the benchmark path are reported, for each transit mode in accordance with procedure:

## 4 CONCLUSIONS

The new semi-automatic procedure for Heavy Vehicles transit authorization (NTEO) allows for the performance of safety checks of all the highway bridges present in the H.V. path quickly, analyzing the effects due to the traffic beyond the influence lines extrapolated by F.E.M. model.

The case study analyzed underlines critical issues in the benchmark path, due to the low level of knowledge adopted and the additional safety factors introduced by the procedure, which further penalize the safety check outcome, especially for the Dapped-End Beams.

However, these additional safety factors have been introduced as they take into account the automaticity of the procedure, providing an additive safeguard of the structures. Moreover, an increase in surveys would reduce the confidence factor and, therefore, it would increase the adequacy coefficient $\zeta_{TE}$ obtained, removing the critical issues.

Furthermore, the data archiving system allows for continuous updating of the conservation status of the structures, but also updating the level of knowledge on materials and construction details, being able to modify the strength of each structural element inserted in the database, for example changing the parameter $\delta$ of residual strength factor.

The accuracy of the data entered in the archive is of paramou nt importance for the correct functioning of the calculation and safety check procedure; therefore, the implementation of an automatic control on the data entered is envisaged, especially for the influence lines, through an order of magnitude comparison with simple notable schemes.

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