Nonlinear Response of Structures to Characteristic Loading Scenarios

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NONLINEAR RESPONSE OF STRUCTURES TO CHARACTERISTIC LOADING SCENARIOS

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Abstract

To assess the safety of an existing bridge, the traffic loads to which it may be subjected in its lifetime need to be accurately quantified. In this paper the 75 year characteristic maximum traffic load effects are found using a carefully calibrated traffic load simulation model. To generate the bridge loading scenarios, an extensive weigh in motion (WIM) database, from three different European countries, is used. Statistical distributions for vehicle weights, inter-vehicle gaps and other characteristics are derived from the measurements, and are used as the basis for Monte Carlo simulations of traffic representing many years. An advantage of this "long-run" simulation approach is that it provides information on typical extreme traffic loading scenarios. This makes possible a series of nonlinear finite element analyses of a reinforced concrete bridge to determine the response to typical characteristic maximum loadings.

Results of the nonlinear analyses are compared to the corresponding results using Eurocode and AASHTO load models.

1 Introduction

Weigh-in-motion (WIM) technology can be defined as a means of weighing vehicles as they travel at full speed on highways. These systems are increasingly being used in an array of application areas such as overload enforcement, transportation planning, bridge and pavement design/assessment and research. WIM systems produce unbiased data due to inconspicuous installation but they have acknowledged measurement inaccuracies. In recent years, the increasing improvement in the quality and availability of WIM data has facilitated a more accurate representation of expected traffic for periods covering many months or even years. This has made possible more realistic probabilistic bridge load modeling which is a main component in bridge safety assessments.

Site-specific bridge assessments, based on measured traffic, offer significant cost reductions for maintenance (O'Connor and Enevoldsen 2009). In order to achieve optimal bridge assessment, it is necessary to estimate, as accurately as possible, the most probable maximum bridge load effects (bending moments, shear forces, etc.) over a given lifetime. The lifetime definition may vary according to the code or specification. For instance, it can be 2 to 10 years for assessment whereas the U.S. AASHTO code (AASHTO 1998) uses 75-year maximum loading for the design of new bridges and the Eurocode (EC1 2003) is based on the distribution of the 50-year lifetime maximum.

Due to the high cost of data collection, measurement periods are usually limited and the quantity of traffic data is generally not sufficient to cover a bridge's lifetime. To extend the traffic data, one approach is to use statistical information from measured traffic to fit a distribution to the load effects and to use these distributions to extrapolate and find the characteristic lifetime maximum values (Nowak 1993; Miao and Chan 2002). An alternative approach is to generate synthetic traffic data, based upon the measured traffic characteristics such as vehicle weights and inter-vehicle gaps, through the use of Monte Carlo simulation (Bailey and Bez 1999; O'Connor and OBrien 2005; O'Brien and Enright 2011). Even with this form of simulation, lifetime maximum load effects usually require some form of statistical extrapolation technique, based on the load effect history. To avoid the problems of extrapolating from short simulation runs it is necessary to run the simulation for a sufficiently long time. These 'long-run' simulations have the added advantage that they provide examples of the types and combinations of vehicles expected to feature in extreme bridge loading events (Enright et al. 2011).

This study presents the results from a Monte Carlo simulation to estimate characteristic maximum load effects based on WIM data collected at three sites: in the Netherlands, the Czech Republic and Slovakia. These characteristic values are compared to the design load effects for the basic load model specified in the Eurocode for bridge traffic loading. It focuses on short to medium span bridges, up to 45m long, where the combined static and dynamic load effects produced by free-flowing traffic are taken to govern. In longer spans, static loading produced by congested traffic has generally been considered to be more critical (Flint and Jacob 1996).

It is evident that the nonlinear nature of reinforced concrete sections will cause some adjustment to the relative values of the bending moments if the structure is loaded into and beyond the service load range (Figure 1). The assumption of elastic behavior is reasonably accurate for low stress levels; but as it approaches its ultimate moment capacity at critical sections, further loading must be taken by other parts of the structure. This means that a redundant structure has an extra reserve of strength beyond that which elastic analysis would predict. Design based on this method calculates the loading required to cause complete collapse. As a result, the bending moment distribution at the ultimate load is quite different from that calculated using elastic theory and will depend on the ultimate moments of resistance of the sections.
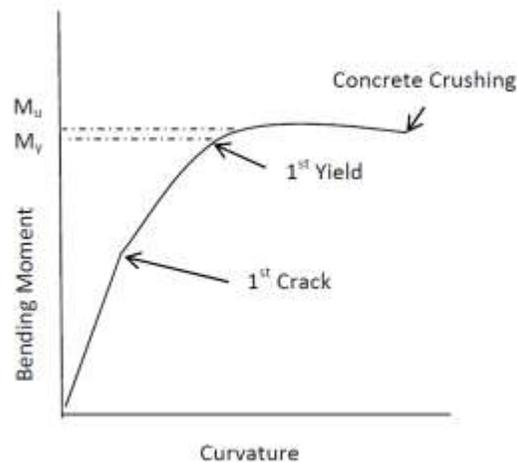
Figure 1. Typical Bending Moment - Curvature Diagram

In the present study, elastoplastic analyses of the structure using the Eurocode and AASHTO Load Models and long-run simulations using WIM data from three different sites has been carried out.

2 WIM data and Monte Carlo Simulation

An extensive database of WIM measurements was collected for trucks weighing 3.5 t or more at the three European highway sites between 2005 and 2008. The WIM measurements are assumed to represent typical highway traffic in each region (OBrien and Enright 2012). These data are collected using embedded piezo-electric sensors in the pavement of the lane. The legal limits for standard (non-permit) trucks are 50 t, 42 t and 40 t in the Netherlands (NL), Czech Republic (CZ) and Slovakia (SK) respectively.

The parameters of each individual truck such as Gross Vehicle Weight (GVW), number of axles and axle spacing are generated using Monte Carlo simulation based on statistical distributions derived from the measured traffic data for each site. Perhaps the most widely used approach is to fit the measured histogram to a multimodal Normal distribution which is known as the parametric approach (O'Connor and OBrien 2005).

This gives a moderately good fit for most of the GVW range but it leads to underestimates of the probabilities in the critical upper tail. A semi-parametric method is proposed by OBrien et al. (2010) to address this issue. It uses the bivariate empirical frequency distribution in the lower GVW range where there are sufficient data and models the upper tail with a parametric fit. This allows the simulation of vehicles heavier than, and which have more axles than, any measured vehicle.

In the simulation model described by Enright (2010) and OBrien and Enright (2011), empirical distributions are used for the maximum axle spacing within each vehicle class. The first axle position at which this maximum spacing occurs varies, and is also modeled using an empirical distribution. Parametric tri-modal Normal distributions are used to simulate axle spacings other than the maximum (which are less critical). The proportion of the GVW carried by each individual axle is simulated using bimodal Normal distributions fitted to the observed data for each axle for each vehicle class. Traffic flows measured at each site are reproduced in the simulation by fitting Weibull distributions to the daily truck traffic volumes in each lane at each site, and by using hourly flow variations based on the average weekday traffic patterns in each lane. A year's traffic is assumed to consist of 250 weekdays.

Enright (2010) describes the use of lane factors based on finite element analyses of typical bridges in order to represent the transverse load distribution on two-lane bridges.

The complete simulation process described by Enright (2010) is developed using a program written in C++, with parallel processing, and by focusing only on significant loading events (i.e. importance sampling). The simulation process has been optimized to allow very long runs to be done, in excess of 1000 years, and this greatly reduces the variability of results and largely avoids issued about the selection of suitable statistical distributions for the extrapolation process.

Annual maximum bending moments from simulations for 10 m, 15 m and 20 m bridges are plotted on Gumbel probability paper (Ang and Tang 1975) in Figure 2. In all cases, the trend is reasonably linear, confirming good agreement with the Gumbel extreme value distribution. The traffic is clearly more "bridge aggressive" in the Netherlands than in the other countries. Results for Slovakia and the Czech Republic are reasonably close, with the load effects being generally higher on the Czech site.
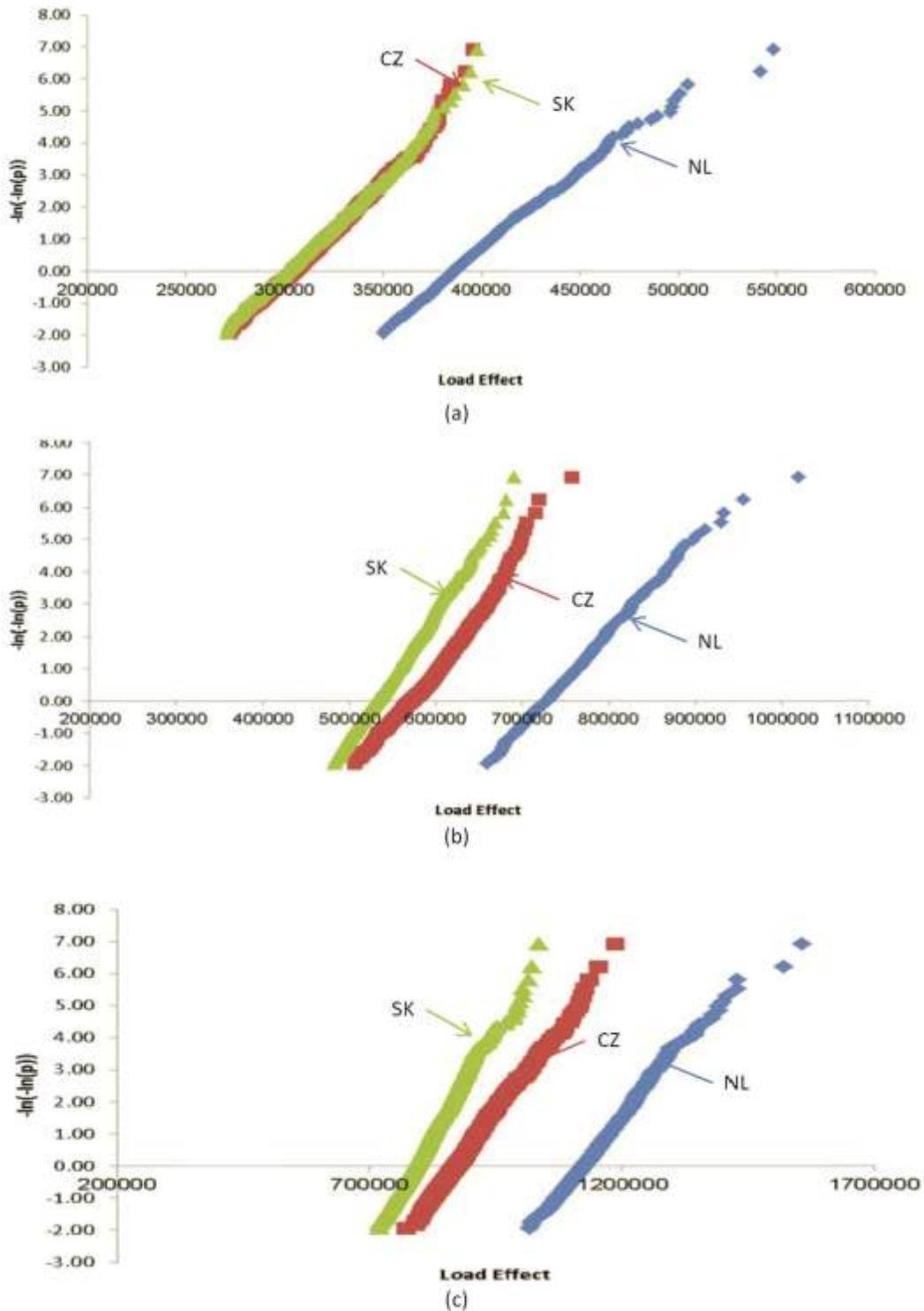
**Figure 2. Annual maximum bending moments from simulation. (a) 10 m, (b) 15m and (c) 20 m
[SK=Slovakia; CZ=Czech Rep.; NL=Netherlands]**

3 Notional Load Models

Notional traffic load models in bridge design codes can generally be divided into two groups: those consisting of a single tandem or truck and those incorporating a uniformly distributed load. The Eurocode

model (EC1 2003) consists of double-axle concentrated loads and a uniformly distributed load which should be applied only in the unfavorable parts of the influence surface, longitudinally and transversally. The Swiss model incorporates a tandem axle group in lieu of the knife-edge load. The Ontario model (OMT 1983) adopts a truck in addition to a uniformly distributed load for long spans. The Australian and USA models (AASHTO) consist of a design truck, tandem and uniformly distributed load for short spans. As measured data was used in its derivation, the Eurocode Load Model 1 ("LM1") (Figure 3) covers most of truck and car traffic and is intended to cover free flowing, congested and traffic jam situations with a high percentage of heavy trucks (EC1 2003).
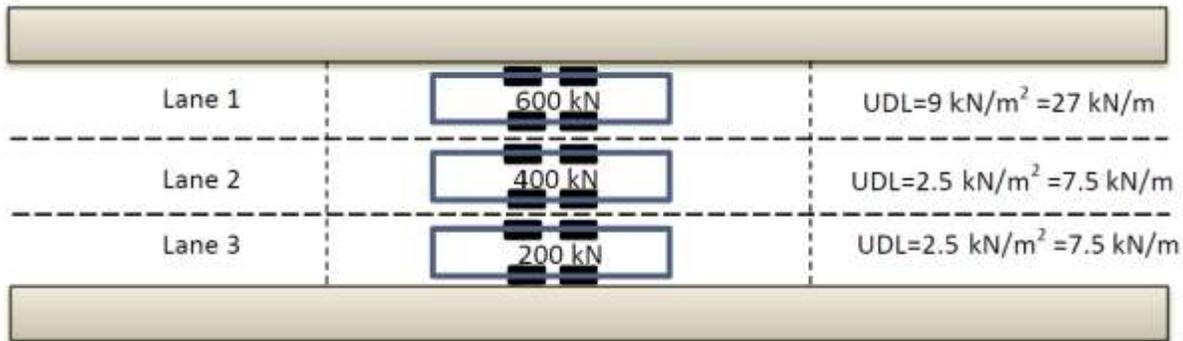

Figure 3. Eurocode load model 1 [UDL=Uniformly Distributed Load]

A bridge with two lanes each 3.65 (total 7.3 m), will have two notional lanes, each 3m wide and 0.65 "remaining area". The load consists of double-axle concentrated loads (tandem system (TS)) in each lane, with axle loads of 300 kN and 200 kN in lanes 1 and 2 respectively. This load model also incorporates uniformly distributed loads, with intensity 9 kN/m^2 for lane 1 and 2.5 kN/m^2 in lane 2 and the remaining area.

The characteristic load effect is defined as that with a 5% probability of being exceeded in 50 years which is equivalent to a value with approximately a 1000 year return period.

The AASHTO vehicular live loading model for bridges was developed as a notional load, HL-93. Although this is not a true representation of actual truck weights, the shear forces and moments resulting from superposition of vehicular and lane loads are deemed to represent the extreme load effects of actual truck events for a 75 year design life. The load model consists of a combination of "Design Lane Load" and either a "Design Truck" or "Design Tandem". The design tandem consists of a pair of 110 kN axles spaced 1.2 m apart and the design lane load consists of a load of 9.3 kN/m uniformly distributed over a 3 m width. Weights and spacings of axles and wheels corresponding to the design truck are specified in Figure 4. The space between the two 145 kN axles is within the range of 4.3 to 9 m.
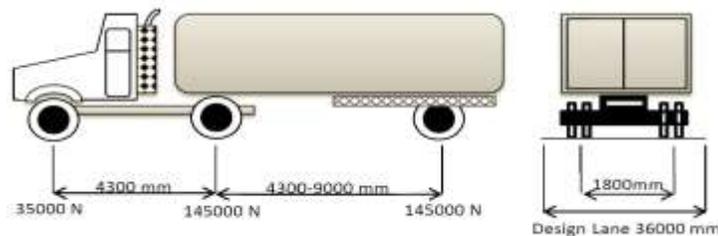
Figure 4. AASHTO design truck

4 Elastoplastic versus Elastic analysis

The importance of ductility in the design of reinforced concrete structures and the ability of a reinforced concrete member to redistribute moment before collapse have been long recognized by structural engineers. Possibly the most important application of ductility is the ability to adjust the structural response to allow for variations in applied load, and to absorb energy during earthquake, blast and other dynamic loadings. The structure behaves in an elastic manner until the elastic limit is reached after which it behaves plastically, as a result of assumed ductile behavior for the relevant sections.

Plastic behavior is governed by the form of the stress-strain curve assumed in tension and compression for the materials of the beam. A material is perfectly plastic if no strain disappears after the removal of load (Megson 2005). Because of plastic behavior at some sections, it is possible for the bending moments to assume a pattern different from that derived from linear elastic structural analysis, so it is necessary to monitor the behavior of the reinforced concrete structure at and near the critical positive and negative moment sections.

According to the theory of plasticity, a structure is deemed to have reached the limit of its load carrying capacity when it forms sufficient hinges to convert it into a mechanism with consequent collapse (McKenzie 2006). For collapse of a structure, three conditions must be met: mechanism, equilibrium and yield conditions. This means that there must be sufficient plastic hinges to develop a mechanism, the bending moments for any collapse mechanism must be in equilibrium with the applied collapse loads and the magnitude of the bending moments anywhere on the structure cannot exceed the plastic moment of resistance of the member, respectively.

A plastic hinge is said to form in a structural member when the cross section is fully yielded. The current study assumes that plastic hinges are concentrated at zero length plasticity. However, in reality, the yield zone is developed over a certain length, normally called the plastic hinge length, depending on the loading, boundary conditions, and geometry of the section. The possible locations for plastic hinges to develop are the points of concentrated loads, at the intersections of members involving a change in geometry, and at the points of zero shear stress for members under uniform distributed load (Chen 1997).

The theoretical analysis in this paper is based on an idealized bilinear moment-curvature relationship for flexural behavior in the plastic region. Furthermore, it is assumed that all sections have the same constant flexural rigidity up to ultimate moment and the moment remains constant at the ultimate value for higher curvatures. The displacements are considered small so that the geometry of the displaced structure does not affect the applied loading system. Material strain hardening is not considered in the current study; consequently a fully yielded cross-section can undergo indefinite rotation at a constant restraining plastic moment, M_p.

In this section, the method employed for elastoplastic analysis of the two span continuous beam of Figure 5(a) is presented. The loading scenario consists of a 2 axle load: 60t and 120t for the first and second axles, respectively. An arbitrary speed of 1 m/s is used with a time step of 0.1 second, and vibration of the beam has not been taken into account Bending moments resulting from applied loads are calculated at each time-step, at all sections along the beam, using an elastic analysis, and are compared continuously with the critical value of bending moment, corresponding to the formation of a plastic hinge.

Bending moments at some points increase as the vehicle moves, and as soon as they reach the critical value at a section, a plastic hinge will form and the remaining load will be redistributed to other parts of the beam. The entire process is repeated for arrange of plastic moment capacity level until the critical level is found.

Plastic hinges for sag moment form at the location of either the first or second axle. As load travels beyond this point, unloading causes the plastic hinge to become elastic again. Therefore, in the following time-step, the superposition of elastoplastic and elastic analyses for these loading and unloading events must be applied. Since a two span beam has only one degree of static indeterminacy, the second plastic hinge will result in a mechanism and collapse of the beam. The plastic moment capacity of the beam of Figure 5 (a) is 3000 kNm.
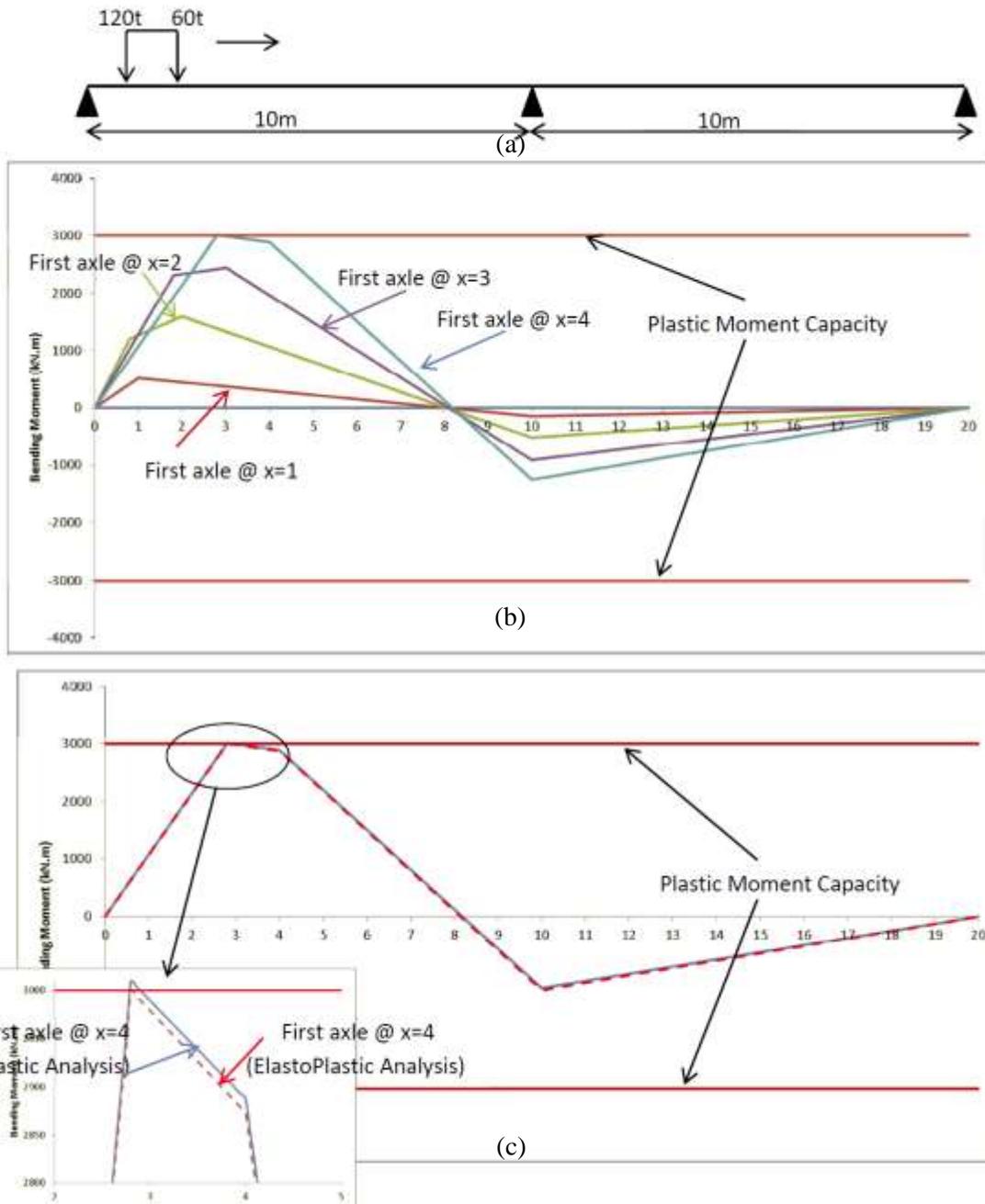
Figure 5. Elastoplastic analysis. (a) Structure, (b) Bending moment diagram for first axle at 1m, 2m, 3m and 4m from the left side (first plastic hinge), (c) Bending moment diagram for elastoplastic analysis when first hinge forms

As can be seen in Figure 5, the location of maximum hogging moment is the central support, and it increases as the vehicle progresses across the bridge. The maximum sagging moment also increases as the vehicle crosses; although it does not have a fixed location, unlike the hogging moment. The first plastic hinge forms at $x = 2.8$ m, corresponding to the second point load position, when the first axle is at $x = 4$ m. This procedure may continue until the second axle leaves the beam. A comparison between elastic and elastoplastic analyses is given in Figure 6.

In the case where the required moment capacity is sought, the trial moment capacity is decreases, in an iterative process, to achieve a mechanism for the most critical load case.
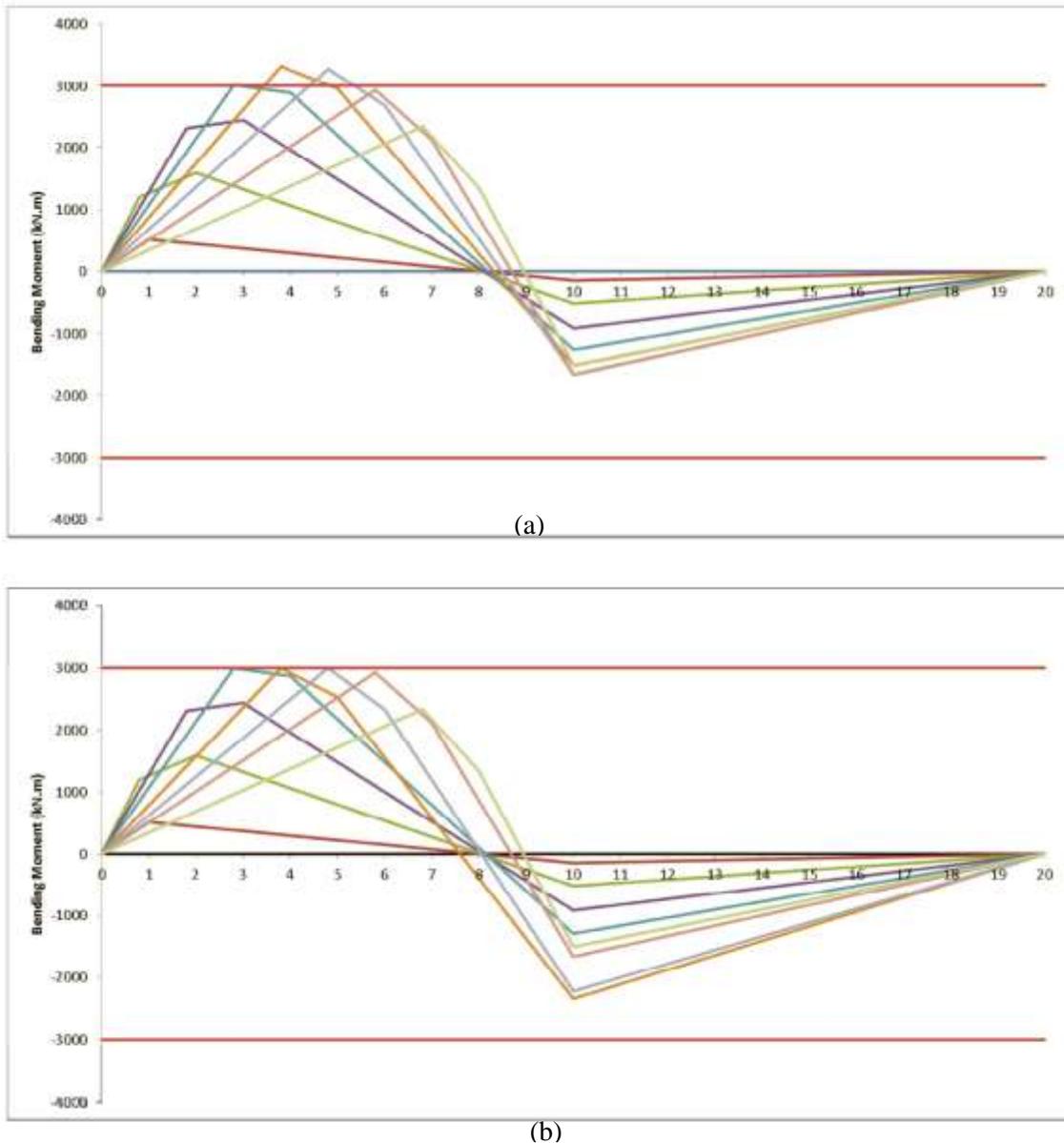

(a)

(b)

Figure 6. Bending moment diagram for first axle at 2, 3, 4, 5, 6, 7 and 8 m from left. (a) Elastic analysis, (b) Elastoplastic analysis

5 Long Run Simulations

Using the elastoplastic method, the minimum required plastic moment capacity has been calculated for a two span bridge under Eurocode, AASHTO and long-run simulation loading scenarios. Three total bridge lengths are considered: 20, 15. As a first step in the study, only extreme scenarios causing annual maximum hogging moment have been used, for each of 1000 years. Twenty extreme scenarios are selected for the three sites as load scenarios (Figure 7). Selection is on the basis that they most closely match the characteristic maximum hogging moment value, based on an elastic analysis.
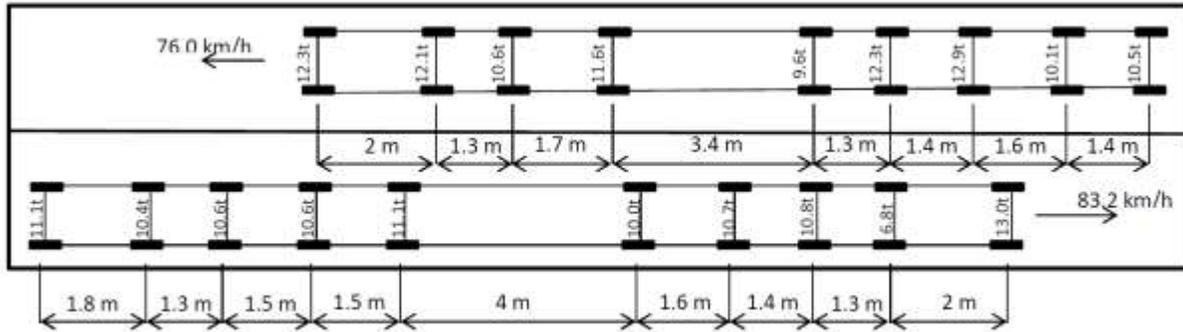

Figure 7. Example of long-run loading scenario (Netherlands 20 m long bridge)

Figure 8 shows the mean and standard deviation of the required plastic moment capacity for the 20 scenarios, comparing the long-run simulation scenarios to results from the Eurocode and AASHTO load models. It shows that, for all the bridge lengths and sites, the elastic and elastoplastic analyses for the long-run scenarios give close results to the AASHTO load model, whereas the Eurocode load model is generally more conservative. For elastic analysis, the AASHTO model using this European traffic, is generally non-conservative but is sometimes conservative when elastoplastic behavior is assumed.
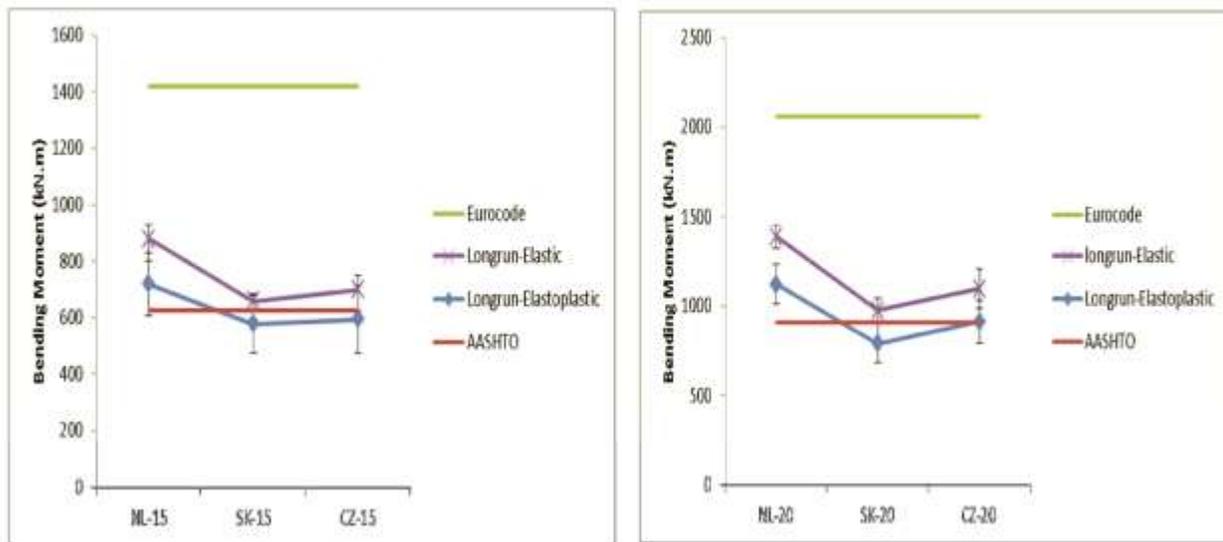

Figure 8. Required plastic moment capacity

6 Conclusion

A moving load nonlinear analysis is performed on a two span beam structure subjected to Eurocode Load Model 1, AASHTO vehicle live loading and extreme long-run simulation scenarios based on WIM traffic. The minimum required plastic moment capacity is found for an indeterminate reinforced concrete beam and compared to the required capacity when subjected to Eurocode and AASHTO deterministic load models. It can be concluded from this study that, as expected, long-run simulation scenarios generally provide less conservative levels of required moment capacity compared to Eurocode. However the AASHTO load model is considerably less onerous and, if results are replicated using American WIM data, it is a cause for concern.

Acknowledgements

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