Vehicles and Gross Vehicle Weight

Vehicles and Gross Vehicle Weight

2

Vehicles and Gross Vehicle Weight

Traffic flow is made up of vehicles that can be sorted into groups, depending on their axle spacings and loads. Vehicles can be considered as standard legal, standard illegal or permit:

* Standard legal vehicles meet the general regulations for axle spacings and weights and do not require a permit, and special vehicles that have exemptions under grandfather provisions.
* Standard illegal vehicles do not meet the regulations of axle spacings, axle and/or GVW.
* Permit vehicles are outside the general regulations for axle spacings and require a permit. They may or may not have this permit and may be in breach of it so they may be legal or illegal.

noindent

Eurocode 1 (86CEN 2003) defines loading due to standard legal and illegal vehicles as ‘normal’ and loading due to permit vehicles as ‘abnormal’. Bridge design load models such as HL-93 and the Eurocode 1 model, are intended to represent the extremes of normal loading, i.e. standard vehicles. Abnormal loading is considered separately – typically a permit is only issued when the bridges on the allowable routes have been rated as have sufficient capacity to carry the specified vehicle.

Countries have legal load limits defined, not only in terms of GVW, but also in terms of axle loads, axle group loads, number of axles, and axle spacings. Usually, a small number of vehicle types tend to govern for characteristic maximum bridge load effects. The load effects for each bridge strongly depends on the distribution of GVW but also on the traffic mix and the dominant vehicle types. From the perspective of bridge loading, the major factors in traffic data are volume, vehicle weights, axle configuration, and multiple presence, i.e., the simultaneous occurrence of multiple vehicles on the bridge, either within a lane or in adjacent lanes.

# 2.1 Categories of Vehicle

There are a great variety of vehicle classification systems around the world. In the US, vehicle classes are specified according to the Federal Highway Administration (FHWA) system (Cambridge Systematics 582007), as shown in 2.1Figure 2.1. Categories of vehicles depend on whether they carry passengers or freight. Non-passenger vehicles are further subdivided according to the number of axles and the number of tractor/trailer units. WIM databases typically provide information about the vehicle's class. Automatic classifiers use an algorithm to interpret axle spacing and total vehicle length information and infer from this the necessary information on the number of units. Vehicles are sometimes mis-classified but it should generally be a small percentage of the total. In most countries, vehicles up to 6 axles (up to Class 9 or 10) are considered standard and Classes 11 – 13 require permits.

Using the FHWA classification scheme, vehicle Classes 9 and 10 tend to dominate on the world’s highways – see 2.2Figure 2.2. This is the 5- or 6-axle tractor/semi-trailer, used for long haul freight transport. It typically consists of a steering axle, a second tractor axle or tandem, and a trailer tandem or tridem group. As many axles in modern trucks are liftable, details of the configuration can change when the driver lifts an axle.

# 2.2 Truck Weight Data

Vehicle weights are clearly fundamental to any consideration of bridge traffic loading. The most accurate systems for weighing trucks are static weigh stations. These are located off-road and typically have static scales built into the pavement. For enforcement systems, heavy vehicles are directed to exit the highway to be weighed – 2.3Figure 2.3. Some of these systems use low-speed WIM which is not fully static but where the speed is very low, and the transverse position is controlled by curbs. In these enforcement systems, an operator then checks if the legal weight limits have been violated. Static weigh stations measure only a tiny fraction of the traffic. Further, their locations are known by truck drivers and illegally overloaded vehicles may try to avoid them, resulting in biased truck weight data.

Portable axle weigh scales, 2.4Figure 2.4, can be set up in any large level area. However, the setup process is time consuming (about 45 minutes per truck) and labor intensive. If the weighing area is on-road, the process can cause traffic obstruction and may increase the possibility of a traffic accident.

At present, the major source of information about the weights of vehicles is the network of WIM systems. While WIM is being used for direct enforcement of overload in some countries (Žnidarič, 3972015), the accuracy of most WIM systems is not sufficient for direct enforcement and they are generally only used for data collection. WIM systems have been collecting good quality data for more than 20 years and, at the time of writing, there are millions of vehicle records at some locations. WIM systems and Continuous Count Stations (CCSs) are the two primary sources of traffic data (169Hallenbeck & Weinblatt 2004). CCSs are also referred to as Counter and Classifier sites or Automatic Traffic Recorders. A continuous count is a volume count derived from permanently installed counters. Counts can be done for 24 hours each day over 365 days per year. There are some different techniques used by CCSs, and the counting and vehicle classification accuracy are subject to the CCS operation environment. Most of CCSs only collect traffic volume, vehicle class and some of them vehicle speed data whereas WIM systems collect this plus the load spectra, i.e., the histograms of axle and gross vehicle weights. WIM data is a powerful first enabler of traffic load assessment and facilitates the development of statistical models of bridge traffic load. Each traffic record includes a detailed description of the vehicle configuration, exact time and date, lane and direction code, speed, GVW, individual axle loads, axle spacings, and class of vehicle (Cambridge Systematics 582007).

One of the first WIM systems was developed in 1952 by the United States Bureau of Public Roads, which was a predecessor of the FHWA (272Norman & Hopkins 1952). It was just a reinforced concrete platform instrumented with electrical resistance strain gauges. The vehicle weight was calculated manually using an oscilloscope attached to the strain gauges. Contemporary WIM systems are very different from the early sensors developed in the 1960s. In addition to axle loads, modern systems determine the vehicle type and they process and transmit the recorded data (8AASHTO 2014). Currently, there are over 700 WIM stations in operation in the United States and thousands worldwide (153Ghosn et al 2010).

There are several factors which can affect the accuracy of measurements collected by any type of WIM system, such as pavement roughness (causing bouncing axle movement or dynamic impact), temperature effects and gradients in the road. ASTM E1318-09 (20ASTM 2009) classifies WIM systems into four types: Type I through Type IV, depending on performance requirements, with Type IV expected to be the most accurate. The COST 323 draft specification (202Jacob 1995) defines a more elaborate framework for the accuracy classification of WIM systems. In general, a Class A system according to COST 323, has 95% of gross weights within an accuracy of ±5%, axle group weights (tandems and tridems) within 7% and single axle weights within 8% . However, the confidence level of 95% is adjusted, depending on the test conditions (number of vehicles, season(s) when tested, whether repeated runs or general traffic).

There is a variety of Weigh-in-Motion technologies available for permanent or temporary traffic data collection. Sensor types include piezo-polymer and piezo-ceramic, piezo-quartz, bending plate, load cell and Bridge WIM (15Al-Qadi et al 2016, 246McCall & Vodrazka 1997). Piezo-polymer and piezo-ceramic sensors come in strips that are placed in a groove in the pavement surface – 2.5Figure 2.5. They are highly temperature sensitive and are mostly used for vehicle count and classification (15Al-Qadi et al 2016). Piezo-quartz sensors also come in strips but are made up of a series of discrete sensing elements. They have a lower sensitivity to temperature fluctuation (386White et al 2006) and belong to the ASTM E1318 Type I systems category. Piezoelectric sensors record a change in voltage in response to the change in pressure due to the passing wheel. They are therefore only effective for dynamic load and are not suitable for static or slow-speed measurements. Piezo-quartz sensors are also pressure sensitive but utilize a quartz crystal force sensing technology. They contain an aluminum alloy profile in the middle of which quartz discs are fitted every 5 cm. All of these strip sensors are installed flush with the asphalt or concrete pavement surface, generally with epoxy adhesive.

The bending plate sensor (15Al-Qadi et al 2016a, 246McCall & Vodrazka 1997), illustrated in 2.6Figure 2.6, consists of strain gauges attached to a steel plate that bends as wheels pass over it. They are generally insensitive to temperature fluctuations. In load cell-based WIM systems, the reactions are measured in the supports of a large plate. The load cells at each support generally use strain gauges to determine the applied forces (15Al-Qadi et al 2016b) and generally provide good accuracy.

There are now numerous WIM stations all over the world, collecting millions of truck records. Researchers have used WIM data from these databases to develop more efficient bridge designs (277Nowak 1999, 281Nowak & Iatsko 2017), to evaluate existing bridges (287Nowak & Tharmabala 1988, 353Sivakumar 2007), and for fatigue studies (140Fisher et al 1983, 225Laman & Nowak 1996, Kulicki et al 2232015).

The changes in truck traffic volume, weight, and configuration in recent decades are reviewed by 17Anitori et al (2017), Ghosn et al (1532010), 235Liao et al (2015), 25Babu et al (2019) and 368Treacy & Brühwiler (2012). A set of protocols and techniques for the collection and analysis of WIM data, along with methods for the calculation of traffic load factors to be used in American LRFD design, are presented by Ghosn et al (1532010).

Virtual Weigh Station (2.7Figure 2.7) is a term used to describe a WIM scale used as an enforcement facility, along with digital cameras and software to process the information in real-time. It does not require continuous staffing and typically it is monitored from another location.

A study in Alabama (Cestel 2020) is used here to illustrate some typical WIM system accuracies. The accuracy of WIM data can be measured in different ways. There are four vehicle parameters of importance: (i) GVW, (ii) axle group weights, (iii) individual axle weights and (iv) weights of axles within a group – see 2.8Figure 2.8. The definition of an axle group varies but is usually a function of the axle spacing – in the Alabama study, a tandem group is defined as two axles less than 1.8 m (6 feet) apart. Axle spacings are also important for studies of bridge loading and the vehicle time stamps, which need to be accurate to 0.01 s to facilitate calculations of inter-vehicle distances.

In the Alabama study example, the accuracy of measurements was checked by a statistical analysis of vehicle parameters using data collected using four different weighing techniques, weigh station, portable scales, WIM, and bridge WIM (B-WIM). The weigh station measurements are taken here as the reference point. The linear correlation coefficient between weigh station and other weighing techniques is presented in 2.9Figure 2.9. For B-WIM the available test parameters indicate the positive correlation coefficients, which vary between 0.64 to 0.81. Lower values of correlation coefficient are noted for the portable scale data – they vary from 0.59 to 0.80 (moderate/high positive correlation). The lowest values of correlation coefficient are obtained from the WIM data – these are in the range from 0.45 to 0.75 (moderate positive correlation).

For dynamic weighing measurements, it is required to verify the data accuracy, and eliminate questionable records, before using it to assess the traffic-induced load effects on bridges. Tolerance checks were carried out on the Alabama data, according to the ASTM E1318-09 (20ASTM 2009) requirements. For a Type I WIM system, these are that 95% of GVW, axle group and individual axle results should be within ±10%, ±15% and ±20% respectively. The WIM system accuracies for GVW are illustrated in 2.10Figure 2.10. The static weigh station weights are used as the reference so results from this system will fall on the diagonal (45o) line. Lines are also shown representing results falling ±10% from the static system. The static portable system is mostly in a range of ±10% of weigh station measurement, but WIM and B-WIM do not meet these tolerances.

# 2.3 Quality Control of Traffic Data

The major source of information about traffic loading is the WIM database and poor-quality data can lead to poor quality bridge load effect calculations. Data errors can result from many issues, including WIM system malfunction, loss of calibration, poor temperature compensation and vehicle miss-classification. Quality Control is an important process of removing suspect data and improving overall data quality.

Two types of error can occur in long-term WIM data collection: random errors (affecting individual vehicles) and systematic errors (occurring frequently and affecting groups of records). There are case studies related to quality checks of traffic data that are adopted by many state agencies in the U.S. (25Babu et al 2019, 121Elkins & Higgins 2008, 357Southgate 1990, 327Ramachandran et al 2011, 324Qu et al 1997, 325Quinley 2010, 223Kulicki et al 2015). However, there is no universal documented state-specific Quality Control procedure. All states gather traffic data as part of FHWA’s Highway Policy Management System and the quality has to meet the minimum requirements prescribed in the guides (325Quinley 2010). Some US states have developed their own Quality Control programs to meet customer needs and achieve maximum performance (378Vandervalk-Ostrander 2009). The need for adequate quality of traffic data in bridge design has been studied extensively in (153Ghosn et al 2010, 355Sivakumar et al 2007).

Important documents that provide guidelines for WIM data Quality Control are the Traffic Monitoring Guide, AASHTO Guidelines for Traffic Data Programs, and the Highway Performance Monitoring System Field Manual (HPMS) (378Vandervalk-Ostrander 2009). In addition, the Long-Term Pavement Performance program collects traffic data as a part of the pavement study (122Elkins et al 2018) and specifies Quality Control procedures for WIM data (382Walker & Cebon 2012, 383Walker et al 2012). More literature related to Quality Control checks in national standards and common practice are discussed in the following sections.

The collected traffic data are recorded using different formats. For instance, in the Traffic Monitoring Guide there is a Station Description format, Traffic Volume format, Vehicle Classification format, Weight format, and five other formats (136FHWA 2016). In the Long-Term Pavement Performance system, different formats can be used depending on the type of software that is used to process the WIM data. The Long-Term Pavement Performance Traffic Quality Control software has 4-card (Classification card) and 7-card (Weight card) data formats. At many WIM locations, the data is processed by vendor’s software that can produce data in a variety of formats. The Traffic Monitoring Guide contains a compendium of Quality Control criteria used by various states and recommends the checks used in the Traffic Monitoring Analysis System. Before data is updated, it is filtered through a number of checks. The AASHTO Traffic Data Program guidelines recommend minimum validation criteria for weight, classification, and vehicle count data. The Long-Term Pavement Performance database has the most rigorous Quality Control checks. Traffic data stored in this database has to comply with the Quality Control checks mentioned in its Information Management System Manual (122Elkins et al 2018, 136FHWA 2016).

In the US WIM data can be utilized for the evaluation of existing bridges (9AASHTO 2018), but the procedure to assess the quality of the WIM data is not discussed in detail. Many filtering criteria have been developed in previous studies to improve the quality of traffic data. Characteristic maximum traffic loading on bridges is determined by an extrapolation process (274Nowak 1993, 195Iatsko 2018). The threshold limits that are used to filter the data may impact the upper tail of the distribution of traffic data that is used in bridge traffic load modeling. Clearly these tail data – the biggest and heaviest vehicles – are highly significant in any assessment of traffic load and their accuracy is key. The importance of the upper tail is discussed more by (289OBrien et al 2010).

An example of a Quality Control procedure is proposed by (25Babu et al 2019) to process and eliminate erroneous records. Firstly, identical rows are sought, which can be an indication of a typical system malfunction. Then, the Quality Control procedure seeks to find errors in the description, vehicle configuration, vehicle weight, speed, and other more advanced checks. The proposed Quality Control checks are shown in 2.1Table 2.1.

The alternative filtering scheme to clean European WIM data is presented by (128Enright & OBrien 2011). An investigation of inter-vehicle (axle to axle) gaps identified several cases where gaps were below 0.2 s. Photographs confirmed these to be the result of trailers being mis-classified as separate vehicles. A comparison of lengths found vehicles where wheelbase (first-axle-to-last-axle length) exceeded total length. This was the result of ‘ghost’ axles – the software typically replicated the rear tandem, making 5-axle trucks into 8-axle vehicles. Based on European WIM data processing experiences, a set of checks was proposed to identify doubtful records. This allows to capture vehicles, with one slightly doubtful attribute to be retained, but vehicles with multiple doubtful attributes to be rejected. The set of rules, along with scores, are presented in 2.2Table 2.2. A score of 7 results in rejection of the record.

# 2.4 Variations in WIM Data

In this section, differences in traffic parameters between the United States and Europe are presented. The traffic is compared for several states in the US and selected European countries. The traffic is very site specific which makes it challenging to develop a consistent live load model for bridge design and evaluation. The WIM data vehicles are compared in terms of GVW, tandem and tridem axle weight.

Differences between European and American extreme vehicles are considered in detail by (230Leahy et al 2014). There are significant differences, perhaps due to the US Federal Bridge Formula and the lower maximum legal weights that exist in the US. Leahy categorizes American extreme vehicles in three types, which he refers to as low loaders, mobile cranes, and cranes with dollies. Low loaders consist of a tractor and trailer and have one large inter-axle spacing, usually in the range, 8 – 13 m. Mobile cranes have a rigid body and closely spaced axles with relatively large axle loads. The mobile cranes in the US generally had fewer axles than in Europe and often rest the boom on a trailing dolly during travel, to allow the crane’s weight to be spread over a greater length. Leahy also found low loaders and mobile cranes in European WIM data but also found crane ballast trucks and did not find mobile cranes with dollies. In Europe, crane ballast trucks often travel with cranes and carry the ballast needed to provide stability to cranes in service. They consist of a tractor and trailer units but do not have the single large spacing found in low loaders. Both mobile cranes and crane ballast trucks have a large load concentrated over closely spaced axles. As such, they are important for short-span bridge loading.

A comparison of vehicle single axle, tandem axle and tridem axle weights in the US (138FHWA 2018) and Europe (293OBrien & Enright 2011) is shown in 2.11Figure 2.11. The histograms for the second axle load are significantly different. The tandem axle distributions are similar with a loaded vehicle weight peak of about 150 kN and unloaded of about 50 kN. For the European tridem axle data, a loaded tridem peak can be seen around 220 kN and unloaded around 60kN. The same trend is absent in the US data which has only one peak around 130 kN.

In the development of live load models, usually the cumulative distribution functions are plotted on normal probability paper to emphasize the upper tails, which include the most extreme vehicles. Series of plots presenting GVW and axle load distribution on normal probability plot are present to emphasize the variation in weight distribution and the importance of maximum load effects from a bridge design point of view.

GVW from several states in the US are shown in 2.12Figure 2.12. These plots are of unfiltered GVW data, so they include permit as well as standard vehicles. The heavy vehicles with GVW above 300 kN, which corresponds to about 30 tonnes, show significant variation. Data from three sites appear to reach an upper limit around 1000 kN (Alabama, Washington, and Rhode Island). This could be explained if the WIM systems did not record vehicles in excess of this weight, a feature of the software used for some WIM systems. Significantly, data from the Florida site does not appear to have reached an asymptote for the quantity of data considered, suggesting that even greater weights may be recorded if more data were analyzed. It should be noted that all WIM data is a function of the size of the database and it is the trend rather than just the particular extreme values that are important. It should be further noted that these upper tails can be strongly influenced by a small number of very heavy vehicles and the trends may not be the same if the survey is repeated. Clearly all of these extreme vehicles are abnormal, i.e., permit vehicles, and are not intended to be represented by a live load model for normal loading.

Tandem axle weight data are plotted on probability paper in 2.13Figure 2.13, where a tandem is defined as two axles spaced between 1.0 m and 2.5 m. There are many 5-axle (Class 9) vehicles in the US that consist of a steer axle and two tandems. It can be seen that the two tandem weight distributions are of similar shape. The maximum tandem weight observed is up to 500 kN.

A tridem is defined here as a combination of three consecutive axles with the distance between the first and third axles between 2.0 and 5.5 m. Tridem weight distributions are plotted on probability paper for various US states in 2.14Figure 2.14. The heaviest tridem loads are observed in Alabama and Florida. The number of tridems identified in the California data is relatively low in comparison to other states, which indicates that such tridem configurations may not be typical in California traffic.

GVW data from five European countries: Slovakia, Poland, Slovenia, the Netherlands, and the Czech Republic, is plotted on probability paper in 2.15Figure 2.15. In Europe, legal weight and dimension limits vary between countries so the data can be expected to be more diverse than for the US. Confusingly, to ensure the free movement of freight, the European Union specifies a minimum value for the maximum legal GVW, i.e., member states cannot impose an unreasonably low upper limit that prohibits standard heavy trucks from passing on their roads. Except for the Netherlands’ site, the distributions are consistent, and it is noted that vehicles in Europe are slightly heavier than in the United States (at least for the sites considered). The Netherlands has great volumes of very heavy trucks, which may be attributed to a dense population in a highly industrialized economy.

Tridem axle weight for Europe are shown in 2.16Figure 2.16. The tridem load distributions are similar for the sites/countries considered. It can be noted that the tridem weights recorded in Europe are lighter than in the United States. The maximum tridem load is below 500 kN, whereas in the US, tridems up to 700 kN were recorded.

# 2.5 Bridge Load Effects

The service life of a bridge depends on many factors such as traffic loads, natural hazards, quality of materials and labor, extreme events, etc. Traffic-induced loads may cause damage to a bridge by fatigue and may accelerate corrosion damage through the periodic opening and closing of microcracks. Every passage of a truck across a bridge creates one or more stress cycles in the structure. For steel bridges in particular, this can result in an accumulation of fatigue damage over time. Fatigue damage increases rapidly when vehicles are overloaded but, perhaps more significantly, overload increases the risk of failure. To maintain bridge safety, the load carrying capacity must resist the load effects corresponding to the specified return period. Overload causes these load effects to increase which, in effect, increases the risk of failure.

A Load Effect (LE) is anything that is affected by load, but the most common LEs considered are bending moment and shear force. The LE due to an axle can be calculated using an influence line, defined as the response to a unit axle force at a point. Knowing the axle weights, the combined LE response to a vehicle is found by simply adding the effects due to each individual axle. As a truck passes over a bridge, it generates a bending moment at each point along the span, and this moment changes as the truck crosses. At the critical point(s), the history of moment during the vehicle crossing is calculated and the maximum value identified.

Mid-span bending moment and support shear force effects are calculated in this way using the WIM data from the selected US states. An example of a 27 m (90 ft) long simply supported bridge is used to illustrate traffic load effects. These LE results are quite distinct from the raw GVW data considered in the previous section as the heavier vehicles tend to be longer so the moment will not increase in proportion to the GVW. The relationship between shear force and GVW is different again, as the shape of the influence line is different. Shear is more strongly influenced by single heavy axles or, in longer spans, by axle groups. Moment on the other hand, is more strongly influenced by GVW or, in shorter spans, by heavy axle groups.

2.18Figure 2.18 shows the moment and shear effects plotted on normal probability paper. It can be seen that Rhode Island has some particularly large bending moments while South Dakota has some quite large shear forces. The largest bending moments and shear forces are caused by vehicles with heavily loaded and closely spaced axles. The same load effects are compared for the European data. In 2.19Figure 2.19 the CDF’s for moment and shear effects are shown. The moments are very consistent for all considered countries. For shear, the effects in Poland are slightly higher than for other countries.

# 2.6 Fatigue Damage

Traffic-induced loads may cause damage to a bridge by fatigue. Every passage of a truck across a bridge creates one or more stress cycles in the structural components, which results in the accumulation of fatigue damage over time. The passage of each heavy truck uses a certain amount of the fatigue life of the bridge. Bridges are subjected to variable amplitude stress cycles. The Palmgren-Miner (259Miner 1954) rule provides a rational method to account for variable amplitude stress cycles. Miner’s rule accounts for the cumulative damage from a spectrum of applied stress ranges of variable amplitude. Using Miner’s rule, an equivalent constant amplitude stress range, referred to as the effective stress range Seff, and can be calculated by:

 (Eq. 2.1)

noindent

where:

ni – number of cycles at the ith stress range, Si,

N – total number of cycles,

Si – constant amplitude stress range.

*m–*fatigue exponent, structural and material dependent.

At a specific point along a bridge, the applied stress range can be determined by dividing the applied bending moment range by the section modulus. Hence, the available WIM data can be used to assess the fatigue damage due to the very large number of stress cycles experienced during the service life of a bridge.

# 2.7 Legal Limits on Vehicle Loads

Traffic consists of legal and illegal standard vehicles and permit vehicles. Legal limits for standard trucks are imposed to ensure the safety of transportation infrastructure and, for pavements in particular, to minimize damage due to heavy axle loads. In the US, federal law prevents states from imposing vehicle weight limits on interstate highways that deviate from established federal weight limits. This means that, for Interstate highways, states are subject either to the standard federal weight limits or to state-specific ‘grandfathered’ limits or exceptions (1US Code §127 1974), i.e., exceptions granted on the basis that they were in existence before the new law was enacted*.* Grandfather provisions, established in 1956, allow exceptions to the federal limits on axle weights and GVW and are particular to each state. Throughout the world, the situation is summarized in 2.19Figure 2.19. Standard vehicles are those that do not require a permit which, in some countries, include vehicles with grandfathered rights. In general, bridge design and assessment codes specify a notional load model for ‘normal’ traffic which is deemed to represent the extremes of standard vehicle loading. Vehicles seeking permits are compared to abnormal vehicles that the bridge has been found to have the capacity to carry.

Permit vehicles are those that require a permit because, according to the regulations on standard vehicles, they are oversized, overweight or both. Permit vehicles need to follow the limitations specified in their permit, which may restrict the gross, single axle, and group axle weights. In the US, states have their own policies on the issuing of permits but must follow federal rules. Permits allow vehicles of specific configurations and sizes to exceed the standard vehicle size and weight limitations. Permits can be issued for single or multiple trips, usually referred to as special and routine permits, respectively. The permit may have limitations on designated routes, the number of trips, times of operation, and the necessity, or not, for escort vehicles. Illegally overloaded vehicles, with or without permits, belong to an unanalyzed portion of bridge traffic load that is more likely to create an extreme loading case.

Vehicle weights and dimensions vary greatly around the world. For example, Europe has very heavy crane ballast trucks which the US does not. On the other hand, the US has mobile cranes with dollies which Europe does not have. These differences between extreme European and US vehicles probably result from the US Federal Bridge Formula, which individual states are required to comply with. The primary purpose of the formula is to distribute vehicle load on highway bridges by limiting the axle configuration and axle load distribution. The formula limits the weight of any set of consecutive axles to:

 (Eq. 2.2)

noindent

where:

*L–*The distance between the outer axles of any group of two or more consecutive axles [m]

*N*–The number of axles in the set under consideration

An exception is that two consecutive tandem axle groups are allowed to carry 15,423 kg each if the overall length of the four-axle set is at least 10 m (36 feet). Grandfathered rights vehicles are also allowed exceptions to the formula.

In the United States, the federal limit on GVW is approximately 40 tonnes. The GVW distribution for selected states is shown in 2.21Figure 2.21, in terms of the ratio of WIM vehicle GVW to the legal limit of 40 tonnes. Hence, all vehicles for which the ratio is above 1.0 can be taken to be permit or illegally overloaded. It can be seen that approximately 5 % of vehicles exceed the legal limit. Florida, Montana and South Dakota include very heavy vehicles that exceed 40 tonnes threshold by three to four times. The Montana site is a notable exception, with 25 % of vehicles above the legal GVW limit. It should be noted that this represents a large number of vehicles per day and increases the probability of multiple heavy trucks meeting or passing on a bridge.

In Europe, the legal limits vary widely between countries. For example, the GVW limit in Poland is 40 tonnes while in Sweden it is 60 tonnes. 2.21Figure 2.21 presents probability paper plots for GVW for the considered European countries, expressed as a multiple of a notional 50 tonnes limit. For most of the considered countries, approximately 1% of vehicles exceed 50 tonnes.

WIM data includes all vehicles and, given the relatively small number of permits issued, will include a small but important number of permit vehicles. In 2.22Figure 2.22 WIM data is compared to the permit vehicle database for Florida. The permit traffic is taken from the permit database that contains only vehicles that purchased and received permits in that state. As expected, the permit vehicles are significantly heavier. For example, at the 0.9999 probability level, the characteristic maximum GVW is around 700 kN for all vehicles whereas, for permit vehicles, it is around 4600 kN. Of course, these weights are not comparable as there are clearly far more vehicles in the general population than in the permit vehicle database. It must also be noted that the extreme permit vehicles tend to distribute the load over many axles.

2.232.24Figures 2.23 and 2.24 show the tandem and tridem axle weight distributions from the WIM and permit databases (FDOT 2020). As expected, axle group weights are heavier in the permit vehicle database, but the difference from the axle group weights in the general vehicle population is much less pronounced than for gross weight. This supports the hypothesis that the US Federal Bridge Formula is requiring the gross weight of extreme permit vehicles to be spread over many axles whose individual weights are not excessive.

# 2.8 Traffic load factors

A rational design or assessment of bridges requires a prediction of the expected maximum traffic load in the specified return period, i.e., corresponding to the specified level of safety. In AASHTO, the specified return period corresponds to the design life of the bridge, which is taken as 75 years (8AASHTO 2014). Thus, in AASHTO, the bridge is designed for the level of traffic load that would be expected to be exceeded just once in its lifetime. In contrast, the Eurocode suggests a design working life of 100 years and a return period of 1000 years so the bridge is designed for the level of load that would be expected in just 10% of bridges in their lifetimes.

For most codes, load and resistance factors are calibrated so that the structure can perform its function for its design life with a probability of failure below the maximum acceptable level. In the current generation of most design codes, the acceptability criterion for the minimum safety margin is specified in terms of a reliability index (278Nowak & Collins 2013) which is an indicator of the probability of failure. Load and resistance parameters are treated as random variables and the reliability index is calculated accordingly.

The main source of information in deriving the statistical parameters of traffic load are the WIM measurements. Although substantial traffic databases have been assembled in recent decades, these are still insufficient to determine the maximum traffic load that a bridge may experience during the specified return period. Therefore, there is a need for an efficient and reliable technique to predict the extreme traffic load effect. Load and load-carrying capacity are both subject to considerable variation. Load carrying capacity, or resistance, depends on material properties and the quality of workmanship, and there are uncertainties involved in the analytical model of the structure. This is why load and resistance are treated as random variables and partial safety factors applied applied to reflect their uncertainty. The derivation of these factors is referred to as calibration of the code. In a reliability-based calibration, the partial factors are chosen that satisfy predetermined reliability criteria, i.e., that provide a reliability index close to the target value.

The most important statistical parameters are bias factor (ratio of mean-to-nominal), coefficient of variation (ratio of standard deviation to the mean) and the type of the cumulative distribution function. However, highway traffic is strongly site-specific, not only from country to country but also within the country and even the local community. In addition, bridge safety is influenced by resistance as well as load so comparison of national design specifications requires a knowledge of not only design traffic load and its factors but also of resistance and its factors.

The first application of a reliability-based calibration procedure for bridge design was the derivation of load and resistant factors for the Ontario Highway Bridge Design Code in Canada (282Nowak & Lind 1979). It was later applied to the calibration of the AASHTO LRFD Bridge Design Specifications (277Nowak 1999). The basis for the current AASHTO LRFD Code (8AASHTO 2014) was developed in the 1980’s (12Agarwal & Wolkowicz 1976, 277Nowak 1999). At that time, there was no reliable truck weight data available for the United States. Since then, an extensive database of WIM data has been collected by US states in multiple locations. The frequencies and weights of vehicles can change over time and this can affect the optimum load and resistance factors.

# 2.9 Limit States and Reliability Index

The current generation of design codes is based on a consideration of limit states. AASHTO defines four types of limit state: ultimate limit state (ULS), serviceability limit state (SLS), fatigue limit state and extreme event limit state. ULS includes moment carrying capacity, shear capacity, axial compression, and axial tension. SLS includes cracking, deflection, and excessive vibration. The fatigue limit state is defined as when the number of load cycles required to reach a limiting value. The extreme event limit state applies to earthquakes and other natural disasters.

Partial safety factors are applied to characteristic/nominal load effects to obtain factored design values. In many design codes, the SLS factors are taken equal to 1.00. For the fatigue limit state, it is important to know not only the magnitude of the traffic load but also the frequency of occurrence (numbers of cycles).

A mathematical representation of the border between acceptable (safe) performance and unacceptable (failure) performance is the limit state function. If R is a variable representing resistance or load carrying capacity and Q is a random variable representing the load effect, then the limit state function can be given by:

 (Eq. 2.3)

noindent

If the probability density functions (PDF’s) for R and Q are as shown in 2.25Figure 2.25, then the PDF of *g(R, Q)* is as shown and the probability of failure is represented by the shaded area. The state of the structure is then determined as safe if *g(R, Q)* is zero or positive and unsafe if it is negative. It follows that the probability of failure, Pf, is the probability of g(R, Q) < 0.

As *g* is the difference of two random variables, R and Q, its mean is the difference of their means:

 (Eq. 2.4)

noindent

where: μR and μQ are the mean values of resistance and load respectively. The standard deviation of the limit state function is:

(Eq. 2.5)

noindent

where σR and σQ are the respective standard deviations. The reliability index, *β*, is defined as (96Cornell 1968),

(Eq. 2.6)

noindent

Hence,

(Eq. 2.7)

noindent

The probability of failure, *Pf*, can be calculated using the CDF of *g*. If R and Q are both normal random variables, then (96Cornell 1968):

(Eq. 2.8)

noindent

where *Φ* is the CDF of the standard normal distribution.

# 2.10 Extrapolation of Imposed Traffic Load Effects

Development of the design load requires a prediction of the maximum expected load effect. Total LE is a combination of load components. The basic load combination for bridges is imposed traffic load, L, and dead load, D:

(Eq. 2.9)

noindent

The statistical parameters for dead load are available in the literature. For in situ concrete, for example, the bias factor is in the range, 1.03 - 1.05 and coefficient of variation is in the range, 0.08 - 0.10.

Finding the statistical parameters for traffic load is more complex. Available WIM data covers much shorter time periods than the specified return periods of, for example, 75 or 1000 years. Therefore, the cumulative distribution functions obtained using the available WIM data have to be extrapolated in some way. In an early study, 282Nowak & Lind (1979) calculated 50-year characteristic maximum bending moment using data from a truck survey conducted by the Ontario Ministry of Transportation (12Agarwal & Wolkowicz 1976). The CDF was plotted on semi-log paper (straight line would imply exponential statistical distribution) to emphasize the trend in the low-probability upper tail region.

In the development of the Ontario Highway Bridge Design Code, the vehicles from the survey data mentioned earlier (12Agarwal & Wolkowicz 1976) were run over influence lines. Mid-span bending moments and shears were calculated from the truck weight data. The simply supported span lengths were varied from 3 to 60 m (10 to 200 ft). Then each LE was divided by the corresponding notional truck loading in the Ontario bridge design code (279Nowak & Grouni 1994). The resulting moments and shears were plotted on normal probability paper. Then, the upper tail of the CDF was extended with an extrapolation line reflecting the trend. The extrapolation turned out to be close to a straight line (279Nowak & Grouni 1994). It should be noted that, in tail fits of this kind, the last few points often deviate randomly from the trend. This is to be expected and is not significant as they represent a very small proportion of the whole data set.

The objective of extrapolation is to predict the expected maximum LE for an extended period of time. In the original calibration of the Ontario Highway Bridge Design Code it was assumed that the economic lifetime of a bridge was 50 years and in calibration of the AASHTO LRFD Code, a lifetime of 75 years was considered. In both of these codes, the return periods were chosen to be equal to the design lives. The maximum value of LE corresponds to 1 – 1/(N+1), where N is the total number of vehicles (records) in the WIM data. If the time between vehicle crossings is t, then for any longer period of time, T, the number of vehicles is N = T/t. For example, if N vehicles are recorded in 1 year, t = 1/N, and, for T = 50 years, the number of expected vehicles is NT, which corresponds to a probability of non-exceedance of, 1 – 1/(NT + 1) on the vertical scale. The expected maximum LE can be predicted by extrapolating the CDF from 1 – 1/(N+1) on vertical scale to 1 – 1/(NT + 1) and then reading the corresponding value of LE on the horizontal scale.

The procedure can be illustrated using WIM data in 2.26Figure 2.26. The number of measured vehicles (number of records) is N = 365,000 and the records were collected for 12 months. The measured vehicles were run over influence lines to determine the maximum bending moment. This was done for simple spans of 9, 27 and 61 m. Non-dimensional moment ratios were obtained by dividing by the corresponding HL-93 moments (AASHTO). The resulting moment ratios are plotted on normal probability paper. For each considered span length, the maximum value corresponds to 1 – 1/(N + 1), where N = 365,000.

The next step involves subjective judgement as the CDF trend needs to be extrapolated. The upper tail of the CDF is extended by what appears to be the best fit. The extrapolated tail of CDF serves as a basis for determining the expected maximum moment for any specified return period, T. For T = 50 years, the probability of exceedance is 1/(NT + 1) = 5.48·10-8 and the non-exceedance probability on the vertical scale is unity minus this value. Similarly, for T = 75 years, the probability of exceedance is 3.65·10-8, as shown in 2.26Figure 2.26. It turns out that the extrapolated upper tails are close to straight lines. The characteristic maximum 75-year moment is then determined as the moment corresponding to the non-exceedance probability of 1 – 1/(NT+1).

Assuming that the occurrence of heavy vehicles is consecutive years is a series of independent events, the CDF of the maximum moment for multi-year time periods can be derived from the CDF for a shorter time period. For example, if F1(x) is CDF for the maximum 1-year moment, then the CDF for the maximum 75-year moment, F75(x) is (278Nowak & Collins 2013):

(Eq. 2.10)

noindent

Ghosn and Moses applied a multi-dimensional stochastic approach to develop a traffic load model based on WIM data collected in Ohio (151Ghosn & Moses 1985). This approach takes into consideration the critical factors that directly affect the expected bridge traffic load such as multi-lane distribution, multiple presence, girder distribution, and future traffic growth factors. The maximum traffic load effect for a 50-year return period is determined as:

(Eq. 2.11)

where:

*a* – factor that depends on truck configuration and span length,

*H* – headway factor,

*W95* – 95% characteristic value for dominating truck weight,

m – variable that reflects the randomness in the axle configuration of representative random traffic.

They estimated the maximum LE (bending moment) acting on a single girder in a 50-year time period, as:

(Eq. 2.12)

noindent

where:

*g* – girder distribution factor and

*Gr –* future traffic load growth factor.

This live load model was applied for assessment of the expected maximum traffic load for evaluation of existing bridges (153Ghosn et al 2010) and for the derivation of state-specific traffic load factors (6AASHTO 2011).

The presented methods for prediction of expected maximum LEs are based on extrapolation and are defined without any reference to a specific vehicle(s): GVW, axle configuration or axle loads. Such a vehicle(s) may be unrealistic or physically impossible. An alternative approach, based on traffic simulation using Monte Carlo techniques, has been applied by other researchers (129Enright & OBrien 2013, 29Bailey & Bez 1999, 292OBrien et al 2006, 309O’Connor & OBrien 2005) and can provide insights into the nature of extreme loading scenarios. Enright & OBrien in particular carried out simulations of thousands of years of traffic to identify the types of loading events that may govern at the level of the specified return periods.

# 2.11 Traffic Load Factors

The design formula in the AASHTO code is typical of what is used in most modern codes:

(Eq. 2.13)

noindent

where:

 load factor

 resistance factor

 nominal load effect

 nominal resistance (load carrying capacity).

The safety margins beyond the characteristic (nominal) level are provided by the load and resistance factors. The role of load factor is to increase the nominal load while the resistance factors decrease the design load-carrying capacity. Load and resistance factors are determined in the reliability-based calibration procedure. The optimum values of factored load and factored resistance can be found as the coordinates of the so-called design point (278Nowak & Collins 2013).

If the limit state function is given by Eq. 2.1, and R and Q are independent Normal random variables, then the reliability index is given by Eq. 2.5. Then the coordinate of the design point for load can be determined from:

(Eq. 2.14)

noindent

And the coordinates of the design point for resistance can be determined from:

(Eq. 2.15)

noindent

where:

 coordinate of the design point for Q

bias factor of R

 coordinate of the design point

 mean value of R

But the factored load is equal to the coordinate of the design point for load, Q\*

(Eq. 2.16)

noindent

So, the load factor is

(Eq. 2.17)

noindent

Load factors for dead load and traffic load can be calculated using the following equations by substituting statistical parameters for load and resistance components.

The coordinates of the design point for resistance can be determined from:

(Eq. 2.18)

noindent

and the resistance factor can be calculated from:

(Eq. 2.19)

It is important to note that the calculation of load factors requires not only statistical parameters of load but also of resistance. Load factor selection cannot be separated from a consideration of resistance. If load and/or resistance are not Normal random variables, then the above listed equations can still be used but the results are approximate.

2.1Figure 2.1 United States FHWA Vehicle Classification Scheme (Cambridge Systematics 582007)

2.2Figure 2.2 Vehicle class distributions at a number of US WIM sites (FHWA classification system)

2.3Figure 2.3 Weigh Station

2.4Figure 2.4 Portable scales weight measurements (397Žnidarič 2015)

2.5Figure 2.5 WIM Strip Sensor

2.6Figure 2.6 Bending Plate WIM system

2.7Figure 2.7 Virtual WIM Station (198IRD 2020)

2.8Figure 2.8 Parameters of a typical vehicle.

2.9Figure 2.9 Correlation coefficient between measured values recorded by Weigh Station and other measuring techniques in Alabama.

2.10Figure 2.10 GVW accuracy of WIM technologies in Alabama test.

2.11Figure 2.11 Histogram of a) Axle 2 load, b) Tandem Axle 4-5 Weight, c) Tridem Axle 3-4-5 Weight based on WIM data in Europe and United States

2.12Figure 2.12 Normal probability paper plots of GVW data in different US states

2.13Figure 2.13 CDF of (a) tandem axle 2-3, (b) tandem axle 4-5 in the US.

2.14Figure 2.14 CDF’s of tridem axle 3-4-5 weights in the US.

2.15Figure 2.15 GVW distributions on Normal probability paper for European sites

2.16Figure 2.16 CDF’s of tridem axle 3-4-5 in Europe.

2.17Figure 2.17 Normal probability paper plots of LE in a 27 m span bridge for a range of US states

2.18Figure 2.18 Normal probability paper plots of LE in a 27 m span bridge for a range of European WIM sites.

2.19Figure 2.19 Vehicle Categories

2.20Figure 2.20 GVW ratios for various US sites on Normal probability paper

2.21Figure 2.21 CDF’s for on Normal probability paper of GVW ratio for various European countries

2.22Figure 2.22 Normal probability paper plots of GVW for WIM and permit vehicle data in Florida

2.23Figure 2.23 Normal probability paper plots of tandem axle weights for WIM and permit vehicle data in Florida, (a) tandem axle 2-3, (b) tandem axle 4-5

2.24Figure 2.24 Normal probability paper plots of tridem axle weights for WIM and permit vehicle data in Florida (a) tridem axle 2-3-4, (b) tridem axle 3-4-5, (c) tridem axle 4-5-6.

2.25Figure 2.25 Probability density functions for load, resistance, and limit state function, *g*.

2.26Figure 2.26 Extrapolation of the upper tail of the CDF of the moment ratio on Normal probability paper

2.1Table 2.1 Quality Control filtering criteria

| **Type** | **Filtering criteria** | **Threshold limits** |
| --- | --- | --- |
| **WIM description** | Station ID | Null or invalid state ID |
| Lane of travel | ≠ (0-9) |
| Direction of travel | ≠ (0-9) |
| **Time stamp** | Invalid year | Null or irrespective year |
| Invalid month | ≠ (1-12) |
| Invalid day | ≠ (1-31) |
| Invalid time | ≠ (0-86399) sec. |
| **Duplicates** | Identical records | Exact copy |
| Same axle weight for consecutive axles | Axle weight = Axle weight n+1= … |
| **Vehicle configuration** | Invalid vehicle class | ≠(1-13) |
| Zero GVW | = 0 |
| Zero axle spacings | = 0 |
| Number of axles | ≠ (2-22) |
| Number of axle weights | ≠ (2-22) |
| Number of axles is equal number of recorded axle weights | Number of axles = Number of axle weights |
| Number of axles spacings | ≠ (1-21) |
| Number of axles is equal to number of axles spacings + 1 | Number of axles ≠number of axles spacing +1 |
| Sum of axle weights is equal GVW ± 10% | ± 10% of GVW |
| Minimum first axle spacing | < 6 ft |
| Minimum axle spacing | < 3.3 ft |
| Steering axle weight | > 40 kips |
| Single axle weight | ≠ (1 - 60 kips) |
| Tandem axle weight | > 60 kips |
| Tridem axle weight | > 80 kips |
| Average left and right wheel weight | > ±20% |
| Total vehicle length | > 220 ft. |
| Speed limits | Vehicle speed | ≠ (10-90 mph) |

2.2Table 2.2 WIM data attribute check with scores (128Enright & OBrien 2011)

| **Attribute** | **Points** |
| --- | --- |
| Rules applied to overall vehicle: |  |
| GVW less than 3.5 t (cars) | 7 |
| Wheelbase less than 1 m | 7 |
| Wheelbase greater than 30 m and first or last axle spacing greater than 10 m | 7 |
| Wheelbase greater than 30 m and speed less than 30 km/h | 7 |
| Wheelbase greater than 40 m | 7 |
| Maximum axle load greater than 15 t and this axle represents more than 85% of GVW | 7 |
| Speed less than 20 km/h | 7 |
| Speed greater than 120 km/h | 7 |
| Speed between 20 and 40 km/h | +5 |
| First axle spacing greater than 15 m | 7 |
| First axle spacing greater than 10 m | +4 |
| Rules applied for each axle: |  |
| Any left or right wheel weight zero or negative | 7 |
| Ratio of left/right wheel weights > 5 | 7 |
| Any axle load zero or negative | 7 |
| Axle load greater than 60 t | 7 |
| Axle spacing greater than 20 m | 7 |
| Points accumulated per axle: |  |
| Ratio of left/right wheel weights between 2 and 3 | +1 |
| Ratio of left/right wheel weights between 3 and 5 | +2 |
| Axle load between 25 t and 40 t | +2 |
| Axle load between 40 t and 60 t | +5 |
| Axle spacing less than 0.4 m | 7 |
| Axle spacing between 0.4 and 0.7 m | +2 |
| Axle spacing between 0.7 and 1.0 m | +1 |