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in
ADVANCED DESIGN OF STRUCTURES

BRIDGE FAILURES:
CAUSES, CONSEQUENCES AND CHANGES IN CODES

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Sessione I

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0. INTRODUCTION

"Those who cannot remember the past are condemned to repeat it". This is true also for bridge construction and therefore my research is focused on finding changing in codes due to important failures.

The compilation of specifications began in 1921 with the organization of the Committee on Bridges and Structures of the American Association of State Highway Officials. During the period from 1921, until printed in 1931, the specifications were gradually developed, and as the several divisions were approved from time to time, they were made available in mimeographed form for use of the State Highway Departments and other organizations.

A complete specification was available in 1926 and it was revised in 1928. Though not in printed form, the specifications were valuable to the bridge engineering profession during the period of development. The first edition of the Standard Specifications was published in 1931, and it was followed by the **1935**(available and analyzed), 1941, 1944, 1949, 1953, 1957, 1961, 1965, **1969**(available and analyzed), 1973, 1977, 1983, 1989, **1992**(available and analyzed), **1996**(available and analyzed), and **1998**(available and analyzed), revised editions, where my research stop. The whole number of editions are seventeen.

In the past, Interim Specifications were usually published in the middle of the calendar year, and a **revised edition was generally published every 4 years**. In United States the development of building codes and standards has become a private-sector enterprise involving federal, state, and local participation. The promulgation of codes is generally based on a **consensus process**; any individual or industry organization may participate in the development of these codes and related deliberations. The purpose is to provide an acceptable level of risk with respect to potential hazards and, at the same time, safeguard the economy. A code or standard do not become law until enacted by the authority having jurisdiction (state, city, etc.).

The *Standard Specifications for Highway Bridges* are primarily specifications that set forth **minimum requirements** which are consistent with **current practice**, and certain **modifications** may be necessary to suit **local conditions**. Factors of safety, load factors, and assumptions regarding interaction between structural components used by designers may or may not reflect *actual* structural performance, but rather may reflect *intended* performance. Provision must be made for both *overload* and *understrength.*

They apply to ordinary highway bridges and supplemental specifications may be required for unusual types and for bridges with spans longer than 500 feet. Specifications of the American Society for Testing and Materials (ASTM), the American Welding Society, the American Wood Preservers Association, and the National Forest Products Association are referred to, or are recognized.

In my research first I have collected all bridges failures occurred in USA, catalogued by year, type, cause, particular details, fatalities, injuries, collapsed part and in what phase. Then I point out new concepts and changes in code due to this failures. There is also a brief resume of the analyzed AASHTO.

1. AIM OF THE WORK

I start with the comparison of the relevant chapters of the Standard Specifications for Bridge Design in AASHTO 1996 and LRFD 1998 Specifications because The two publications contain the specifications to be followed in the United States in matter of bridge design. Then I go to the older AASHTO1935 and 1969 to capture what are the main changes, and to understand what was the lesson learned from a failure, that brought to important changes in code.

Although only two years elapsed between the publication of these two Specifications, there are significant differences between the two texts both at general and specific level. First of all, just looking in general at the way the two Specifications are written, it is possible to note that the 1998 version is much richer in content than the 1996 one. In fact, more formulas for the different verifications to be performed are provided as well as more coefficients and indications about how to perform the verifications. Moreover, in the 1998 text, specifications are explained and clarified by means of a commentary regarding all the considered subjects. As a consequence, the new Specifications are more detailed than the previous ones.

However, it has also to be pointed out that, due to the abundance of specifications provided and to the fact that these specifications are generally performance based (whereas those contained in the 1996 text are prescriptive), the new version leaves more room for misinterpretations with respect to the 1996 Specifications, that are simpler and more logical.

2. HYSTORY

1. Design philosophies

Since approximately 1931, the bridge design standards prescribed by the American Association of State Highway and Transportation Officials have followed a design philosophy called Allowable Stress Design (ASD).

Although the detailed specifications have been periodically revised, the philosophy underlying the ASD code has remained the same. Developed with metallic structures in mind, the design methods are based ***upon elastic behavior***. Allowable stresses are calculated by dividing the material yield or ultimate strength by a safety factor. The safety factors are subjectively defined, conservative values that attempt to account for the uncertainty in the design of highway bridges.

In the 1950s, as extensive laboratory data on failure mechanisms of structures began to accumulate, researchers recognized some weaknesses inherent in the concepts of the ASD code. Allowable stress codes do *not permit* design directly against the *actual failure limit states*, unless those limit states occur within the elastic range. This limitation applies for all materials where inelastic behavior occurs at the onset of failure. Even nominally isotropic homogeneous materials such as steel do not behave in a linearly elastic manner in the failure region, either as a consequence of material nonlinearity, or instability, or some combination of the two. Thus a limit state design approach is preferable.

The first generation of AASHTO code to use a limit state method for design of steel structures, called *load factor design (LFD),* was introduced in the 1970s (see chapter AASHTO 1969) as an *alternative* to the ASD specifications. The LFD specification retained the ASD load model and *did not consider differing levels of uncertainty in structural resistance models*, but for the first time permitted design directly against the failure state, instead of against fictitious allowable stress states.

In addition to its failure to adequately address failure limit states, the ASD design approach does not provide a *consistent measure of strength through the use of probabilistically derived safety factors*, which is a more suitable measure of resistance than is a fictitious allowable stress. In particular, since the safety factors are only applied to resistance, the differing levels of variability in the various load components cannot be adequately taken into account within the ASD format.

The effort to incorporate LRFD, resulted in the first edition of the AASHTO LRFD Bridge Design Specifications (AASHTO, 1994). This LRFD bridge design code was adopted with a provision to consider phasing out the ASD specifications in the near future. The second edition was introduced in 1998, and the third edition became available in July, 2005

In addition to new ***reliability based design philosophies***, and the introduction of a *more sophisticated limit state approach* to categorizing structural resistance, the AASHTO 1998 contained numerous changes in loads and load applications, when compared to the AASHTO 1196ASD code. These changes include a revamped live load model, a newly derived set of load distribution factors, and a new, and slightly more conservative, set of *impact factors* (now called *dynamic load allowances*).

Comparing the Specifications for Bridge Design in AASHTO 1996 and the LRFD 1998 Specifications, it is possible to note that many important changes and some improvements have been done. In particular, the most relevant change is represented by the introduction of loads and resistance factors (LRFD) in the new specifications, and the transition from allowable stress (ASD) to ultimate load method.

The loads and resistance factors are very innovative because they introduce the new concepts of *uncertainty* and *reliability*, changing the deterministic bridge design philosophy of the previous AASHTO specifications.

In fact, the AASHTO 1996, written by the AISC (American Institution of Steel Construction), can be defined as a prescriptive and descriptive code with directions and rules.

It applies the allowable stress method (ASD), that combines the loads by the use of established safety factors. All this makes the AASHTO 1996 simple to apply, very logical and clear, but it, actually, does not take in account the complex interactions among loads and structure and seems to be pessimistic and limitative. This is because of the assumption of fixed safety margins and other simplifications also due to the lack of more recent powerful computational tools that now allow complete and sophisticated analysis. Basically, it is possible to see looking in the following definition

$$\sum Q_i \leq \frac{R_n}{FS}$$

where $\sum Q_i$ is the *required strength,* which is the summation of the *load effects* Q_i (i.e., forces or moments), and $\frac{R_n}{FS}$ is the *design strength,* which is the *nominal strength or resistance* R_n*,* divided by a *factor of safety* FS*. T*he allowable stress design method assumes that the ultimate limit states will automatically be satisfied by the use of allowable stresses. Depending on the variability of the materials and loads, this assumption may not always be valid. Moreover in this method it is assumed that all loads have the same average variability, because taken into account in the same FS. Allowable stress design very often come out to include the inability to properly account for the variability of the resistances and loads; lack of knowledge of the level of safety; and the inability to deal with groups of loads where one load increases at a different rate than the others. The latter condition is especially serious when a relatively constant load such as dead load counteracts the effects of a highly variable load such as wind.

In the new specifications LRFD 1998, written by the ACI (American Concrete Institute), *safety factors* have been replaced by factors that are calibrated and based on stochastic analysis, empirical data and structural reliability1 theory to describe the uncertainty of loads and resistance, and nonlinearity of materials. The introduced concept of the *uncertainty* is very powerful and allows to design in order to maximize the expected useful life of structures, improve maintenance characteristics using more accurate analysis methods. Uncertainty is basically associated and due to ignorance, randomness and vagueness2. It has been taken in account by including safety parameters and indexes in the loading combinations. Those parameters are derived by the above-mentioned reliability theory based on a representation of the demand of design loads and the supply of resistance using normally distributed functions.

In addition, to load factors that increase load's values, the LRFD Specifications introduce also resistance factors that reduce the *actual resistance capacity* of the structure

$$\varphi R = R_n \geq \sum \eta_i \gamma_i Q_i$$

The final value of the resistance is called nominal resistance R_n, and this is a significant change with respect to the previous code (1996) that accepted to satisfy the relation: $R \geq 2Q$. The safety factor 2 was chosen taking in account the (uni-axial tensile strength) behavior of linear, uniform and isotropic materials (mainly steel, because the code was written by AISC). On the contrary, the new code refers also to behaviors of *nonlinear* or *non-homogeneous*

[1] The structural reliability is the probability that a structural system is operational after the application of certain loads and external conditions.

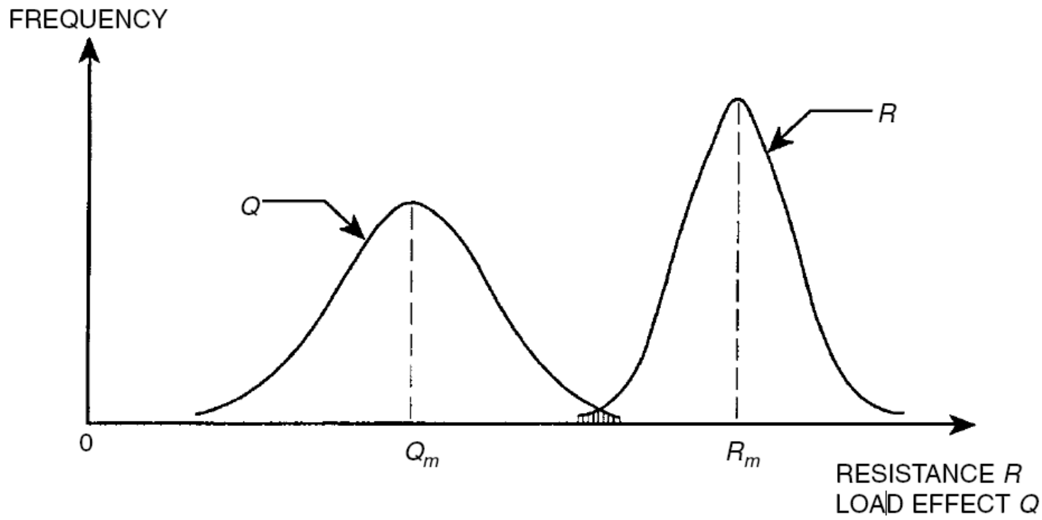
[2] Ignorance is the primary source of uncertainty and is due to insufficient data, like measurements about the features of materials and structural behaviors. The randomness is related to unexpected nonlinear behaviors or variations of the properties of materials and structures. Finally, vagueness implies fuzzy sets because of qualitative, unclear or non specific input data or approximated models.

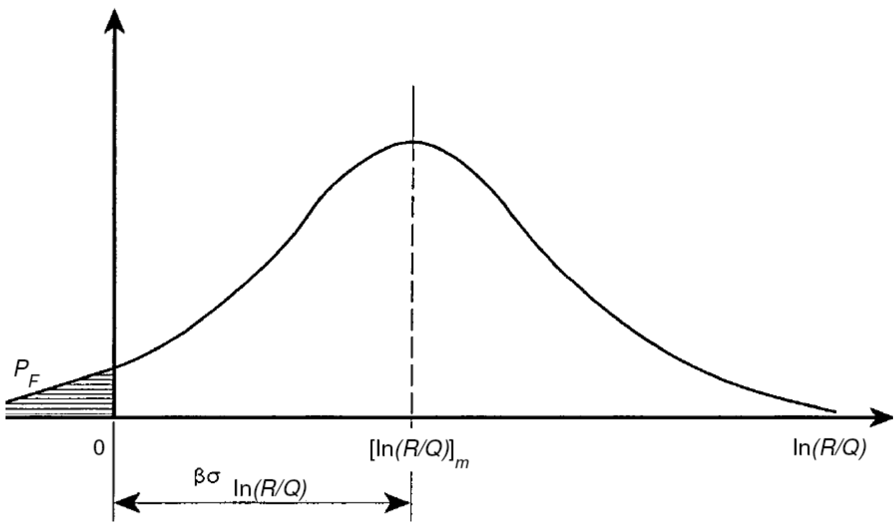
materials, like concrete; for this reason, probabilistic considerations are developed for making acceptable the relation between *loads* (demand) and *resistance* (supply).

Finally, another main aspect of the design philosophy of the new code is the presence of several *performance levels* defined by *limit states*. Thanks to all this characteristics, this performance based code can accomplish and satisfy more requirements for a good design of a bridge but, at the same time, it becomes more complex, sometimes ambiguous, and is often misinterpreted by designers and engineers.

Limit states are generally defined as those conditions of a structure at which it ceases to fulfill its intended function, and can be divided into two categories, *strength* and *serviceability.* Strength (i.e., safety) limit states are such behavioral phenomena as the onset of yielding, formation of a plastic hinge, overall frame or member instability, lateral-torsional buckling, local buckling, tensile fracture, and development of fatigue cracks. Serviceability limit states include unacceptable elastic deflections and drift, unacceptable vibrations, and permanent deformations. Design criteria should ensure that a limit state is violated only with an acceptably small probability, by selecting the load and resistance factors and nominal load and resistance values that will never be exceeded under the design assumptions.

Both the acting loads and the resistance (strength) of the structure to loads are variables that must be considered. In general, a thorough analysis of all uncertainties that might influence achieving a *limit state* is not practical, or perhaps even possible.





In particular, the randomness is taken into account by means of the *reliability function*:

$$\beta = \frac{R^* Q^*}{\sqrt{{\sigma_R}^2 + {\sigma_Q}^2}}$$

Where:

- $R^*$and $Q^*$are the mean values of the Gaussian variables R (supply) and Q (demand) respectively;
- ${\sigma_R}^2 + {\sigma_Q}^2$ are the variances of the Gaussian variables R (supply) and Q (demand), respectively.

The reliability function is directly related to the γ coefficients, which multiply the acting loads. Therefore, by means of β the random nature of supply and demand is accounted in the verification of the structural elements. Moreover, the possibility to set the value of β according to the considered case allows for a greater flexibility in the design process. These elements makes the LFRD method more accurate with respect to the approaches provided in the 1996 Specifications.

Now let's summarize the main changes in the AASHTO editions

AASHTO (1939)	LRFD (1969)
Prescriptive and descriptive	Prescriptive and descriptive
Deterministic	Deterministic (first concept of levels of uncertainty)
Very very Simple	Simple and logical
Safety	Safety
Allowable Stress Design (ASD)	Allowable Stress Design (ASD)+(LFD)
No consistent measures for strength with probability safety factors	Load combination with design against Failure limit state

AASHTO (1996)	LRFD (1998)
Prescriptive and descriptive	Performance based
Deterministic	Uncertainty/ Probabilistic analysis
Simple and logical	Complex and often misinterpreted
Safety	Reliability
Allowable Stress (ASD)+(LRFD)	Load and Resistance Factors (LRFD)
Load combination with more sophisticated limit state approach	Limit states

The structural reliability is the probability that a structural system is operational after the application of certain loads and external conditions.

Ignorance is the primary source of uncertainty and is due to insufficient data, like measurements about the features of materials and structural behaviors. The randomness is related to unexpected nonlinear behaviors or variations of the properties of materials and structures. Finally, vagueness implies fuzzy sets because of qualitative, unclear or non specific input data or approximated models.

2. Design loads

DEAD LOADS:

The first notable difference is in the definition of *the dead load*. In AASHTO 1935 the dead load is simply considered as the weight of the structure complete (weight per cubic foot give)

Steel	490
Cast iron	450
Aluminum alloys	175
Timber (treated or untreated)	60
Concrete, plain or reinforced	150
Loose sand and earth	100
Rammed sand or gravel, and ballast	120
Macadam or gravel, rolled	140
Cinder filling	60
Pavement, other than wood block	150
Railway rails and fastenings (per linear foot of track)	150

The snow and ice are not considered because their presence decrease the other live loads. No different safety factors are applied.

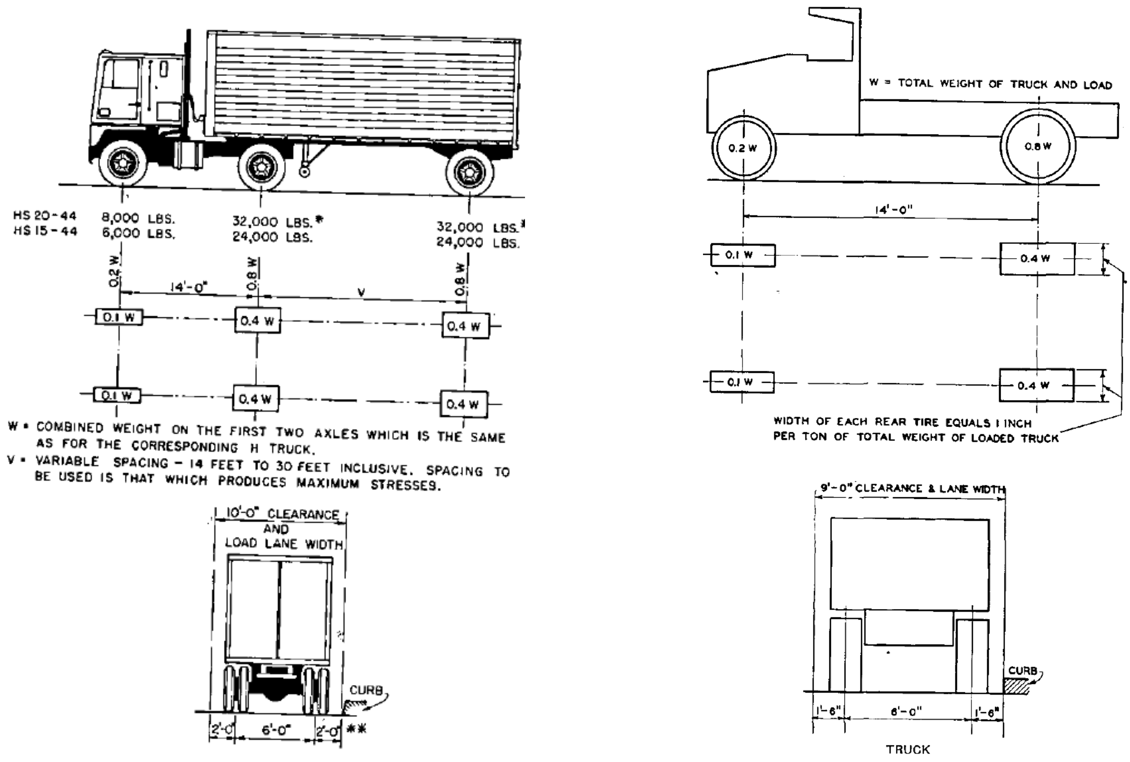
In AASHTO 1969 the dead loads are defined in the same way of the AASHTO 1935 and also the material have the same property except for the timber.

	Weight per cubic foot, pounds
Steel or cast steel	490
Cast iron	450
Aluminum alloys	175
Timber (treated or untreated)	50
Concrete, plain or reinforced	150
Compacted sand, earth, gravel or ballast	120

In particular,comparing AASHTO 1996 and 1998, although both the versions define the dead load of a bridge as the weight of the entire structure as well in 1935 and 1969, including all the elements, such as roadway, sidewalks, cables, utilities, exc., the 1998 Specifications provides an entire paragraph about this dead load. In particular, factors such as *water content, compaction, earthquake effects, overconsolidation* are considered and specifications about *active and passive pressures and friction angle* are included.

LIVE LOADS:

Dating back to the 1935s, AASHTO had used two basic live load, defined as weight of the applied moving load of vehicles, cars, pedestrian, models to approximate the vehicular live loads that would be experienced by a bridge: the *H design trucks*, H20, H15 H10 where H15 and H10 are respectively the 75 and 50n% of the H20 loading, and the *design lane loading*.



STANDARD HS TRUCKS

In the 1969s, AASHTO had used three basic live load models to approximate the vehicular live loads that would be experienced by a bridge: the *H and HS design trucks*, the *design tandem*, and the *design lane loading*. The HS design truck, or standard truck, is a three-axle truck intended to model a highway semitrailer.

The design tandem is a two-axle loading intended to simulate heavy military vehicles. The design lane loading primarily consists of a distributed load meant to control the design of longer spans where a string of lighter vehicles, together with one heavier vehicle, might produce critical loads. On the other hand, in the AASHTO ASD codes 1996 and 1998, these three type of live loads are the same, but each of these load models was applied individually.

Subsequently, AASHTO used the results of the truck data, together with a statistical extrapolation to a 75-year design life, to provide the basis for the AASHTO LRFD design loading. The results of researchers of these studies indicated that the AASHTO ASD live load models consistently underestimated the load effect of vehicles on the road today.

AASHTO LRFD 1998 and 1996 contains two live load models with the design lane superimposed upon them, the *design truck* and the *design tandem*. This is expected to produce significantly increased design loads.

The second notable change in vehicular load modeling between the ASD and LRFD codes is the method used to approximate the *live load amplification* due to *dynamic loading*. The dynamic load allowance, formerly referred to as the ***impact factor***, is an *equivalent static magnification factor to be applied to a statically applied load on a structure in order to predict the additional response amplitude resulting from the motion of the load across the structure*. In a highway bridge, the actual dynamic response amplitude is a function of a number of factors. Such factors include, but are not limited to bridge span, type (continuous or simple span), number of girders, slab stiffness, bridge damping, deck roughness, vehicle mass, vehicle velocity, damping, number of axles, suspension system, vertical velocity upon entering the bridge, probability of coincidence of maximum load and maximum impact, and position of load relative to a girder.

In an attempt to simplify this dynamic behavior, the AASHTO ASD specifications provided an impact factor that varies with the length of the bridge, but is to be no greater than 0.3. The new LRFD specifications simplify the model even further by providing constant dynamic load allowances: 0.33 for *strength limit states of all members*, 0.75 for *deck joints*, and 0.15 for *fatigue.* The highway live load has to be increased by an impact factor. This is when the highway live loads are applied to superstructures, piers, portions above the ground line of concrete or steel piles that supports superstructures. The amount of the above mentioned impact allowance or factor is expressed in terms of a fraction of the live load stress:

$$I = \frac{50}{L + 125} \leq 30\%$$

where I is the impact fraction and L is the length in feet of the portion of the span that is loaded to produce the maximum stress in the member.

In AASHTO 1969 it is only written to take into account dynamic loads, such as centrifugal force, Lateral forces for moving live loads; centrifugal force 10% of the live load, uplift in the leeward traffic lane with a constant value of 400 pounds per linear foot of lanes and 800 if there is also railway traffic.

The manner in which loads are transmitted to each girder is a third modeling consideration that was revised with the adoption of the LRFD code. The response of a bridge to a passing vehicle is a complex deformation, in which a portion of the load is transferred to each of the supporting girders. The exact proportion of the load carried by each girder is a function of the girder spacing, span length, slab stiffness, the number and locations of cross frames, and the placement of the load on the span. Both the AASHTO ASD and LRFD codes permit the use of *distribution factor methods to model the transfer of loads through the slab to each girder.*

The AASHTO ASD distribution factors were originally developed using orthotropic plate theory, and the resulting equations are based upon the girder spacing alone. These ASD distribution factors are plagued by *inconsistency*, sometimes being overly conservative and at other times being non-conservative.

In the LRFD the vehicular live load is given by the *combination* of the *design truck* or *design tandem* applied together with the design lane load.

The LRFD design truck is identical to the Standard HS20 truck (AASHTO 1996). Then, there is also the design tandem, that consists of two axels (25 kip each) spaced 4 ft apart. In either case, the transverse spacing of wheels is taken as 6 ft. The LRFD design lane consists of a uniformly distributed load of 0.64 klf in the longitudinal direction; it is distributed transversely over a 10ft width. The static effects of the design truck or design tandem are multiplied by $(1 + IM/100)$, where IM is the *dynamic load allowance*. It takes in account the dynamic effects and replaces the previous Impact fraction of the AASHTO 1996. For all limit states, except Fatigue, IM is taken as 33%; instead, for *Fatigue Limit State*, it is taken as 15%. This dynamic load allowance is not applied to the design lane load, but only to the design truck or tandem.

In the LRFD 1998 Specifications, it is important to note that to consider each possible combination of number of loaded lanes is considered by multiplying the vehicular live load by the corresponding *multiple presence factor*.

# of loaded lanes	**multiple presence factor m**
1	1.2
2	1.0
3	0.85
>3	0.65

For the AASHTO 1996, a similar provision is taken in account if maximum stresses are produced in any member by loading a number of traffic lanes *simultaneously*. In those cases, a percentage of the live loads have to be used.

# of loaded lanes	Percentage
One or two lanes	100
Three lanes	90
Four or more lanes	75

For the AASHTO 1969

# of loaded lanes	Percentage
One or two lanes	100
Three lanes	90
Four or more lanes	75

For the AASHTO 1935

# of loaded lanes	Percentage
One or two lanes	100
More than two	-1% of each ft>18ft > 75

Furthermore, another difference is found between the AASHTO 1996,1969,1935 and 1998 codes in the definition of the *wind load*. In the AASHTO 1996 (ASD) Specifications the wind load is a *uniform distributed load applied to the exposed area of the bridge*. The wind load may be reduced in intensity in function of a map of wind speeds for the U.S.

In the LRFD Specifications in AASHTO 1998 many factors have to be calculated for the *wind load*. The *wind direction* has be varied to determine the *extreme force effect* in the structure. The *velocity* and the *pressure* of the wind on the structures are determined by the use of specific formulas in function of several factors, also *applying the design wind pressure on both structure and vehicles when vehicles are present.*

In 1935 AASHTO the wind force is simply *30 pound per square foot on* $1\frac{1}{2}$ *times the area of the structure* as seen in elevation, including the floor system and railing and one half the area of the trusses or girders in excess of two in the span

In 1969 AASHTO the wind load is 100psf on all the elements in the orthogonal direction of the bridge; then depending on the type of structure, the angle of wind direction is changed as shown as follow

Skew Angle of Wind (Degrees)	Trusses: Lateral Load Per Sq. Ft. of Area (Pounds)	Trusses: Longitudinal Load Per Sq. Ft. of Area (Pounds)	Girders: Lateral Load Per Sq. Ft. of Area (Pounds)	Girders: Longitudinal Load Per Sq. Ft. of Area (Pounds)
0	75	0	50	0
15	70	12	44	6
30	65	28	41	12
45	47	41	33	16
60	25	50	17	19

Skew Angle of Wind (Degrees)	Lateral Load Per Lin. Ft. (Pounds)	Longitudinal Load Per Lin. Ft. (Pounds)
0	100	0
15	88	12
30	82	24
45	66	32
60	34	38

Another change has been done for the definition of the ***earthquake load***. The AASHTO 1996 considers the earthquake load as an *equivalent static horizontal force* applied at the *gravity center* of the structure (EQ) and prescribes to use response spectrum dynamic approach, when a seismic analysis is necessary for complex structures.

In the AASHTO 1998 LRFD Specifications the earthquake loads are defined as *horizontal force effects* determined on the bases of the *elastic response coefficient*, and the *equivalent weight of the superstructures*, and then *adjusted by the response modification factor R*. The seismic force is defined in function of the *seismic performance zone*, the type of *soil* and other response modification factors. Then, the components of the seismic force in each direction are combined using the Complete Quadratic Combination. Dynamic analysis as the *dynamic modal analysis*, the *time history* analysis are taken in account by the LRFD 1998.

In 1935 AASHTO edition there are no seismic loads to take into account; also in the voice “other loads when they exist”, seismic loads are not written.

In 1969 AASHTO edition appear one of the first earthquake stress concept as follow

1.2.20. “In region where earthquake may be anticipated, provision shall be made to accommodate lateral forces from earthquake as follow

$$EQ = CD$$

Where

EQ is the lateral force applied horizontally in any direction at the center of gravity of the weight of the structure.

D is the dead load of the structure

C is a coefficient (0.02 or 0..04 or 0.06) depending on the type of foundation. Live load can be neglected.

In recent year methods of designing seismically safe bridges and analyzing their seismic failure probability have progressed rapidly. These development have often been initiated by failure occurrence.

- Superstructure must be tied to their supports and foundations in such a way that superstructure cannot fall off substructures during seismic activity. The failure of the two-level San Francisco Oakland Bay Bridge during the Loma Prieta earthquake in 1989, when a section of the upper road deck fell onto the lower, was due to the fact the bridge had been completed in 1957, before the implementation of this new regulation.
- The bridge foundation must be designed in such a way that damage through ground liquefaction cannot occur

After the earthquake in the Northridge area of Los Angeles in 1994 bridge column design had to include:

- Careful design and construction of the links in column reinforcement
- Now seismically safe design is at least as important as analysis.

Finally, it is relevant to note that several kinds of *transient load* are included in the LRFD Specifications 1998. In fact, the *vehicular braking force* BR, the *creep* CR, the *vehicular collision force* CT, the *vessel collision* force VC, the *pedestrian live load* and the *water load*, all of them are not considered in the AASHTO 1996, 1969, 1935, instead in the LRFD 1998 is specified how to consider and apply those loads to the structure of the bridge.

After a failure of Takoma bridge, many studies were done and also new paragraph (after a more detailed wind pressure values case by case) appear in the new AASHTO editions for example in AASHTO 1969

c) **overturn force:** the effect of forces tending to overturn structures shall be calculated and there shall be calculated under group II and group III of the combination below, and shell be added an upward force applied at the windward quarter point of the transverse superstructure width. This force shall be 20 pounds per square foot of deck and sidewalk plan area per group II combination and 6 pounds per square foot for group III combination. The wind direction shall be at right angles to the longitudinal axis of the structure.

Group II and Group III are those combination where there is the wind load on the structure

Group II: D+E+B+SF+W

Group III: D+L+I+E+B+SF+LF+F+30%W+WL+CF

Where: D=dead load

L=live load

I=impact load

E=Earth pressure

B=Buoyancy

W=wind load on structure

WL=wind load on live load – 100 pounds per linear foot

LF=longitudinal force from live load

CF=centrifugal force

SF=Stream flow pressure

In AASHTO 1935 there are no explicit concepts/paragraph that explain what are the range of serviceability vertical displacements/uplifting; After **Mississippi in Chester, Illinois 1944,** where the failure occurs due to **excessive uplifting wind load** not considered, in the 1969 edition there is explicitly a new paragraph 1.2.16 "provision shall be made for adequate attachment of the superstructure to the structure should any loading or combination of loading, increate by 100 per cent of the live-plus-impact load, produce uplift at any location

AASHTO (1939)	LRFD (1969)
Live load: H+Design lane	H+HS+Design lane+Design tandem
LL Uniformly distributed	LL Uniformly distributed
No Impact Factor	Impact factor only dependent on Length of the span and <0.3
Wind load:30psf	From 18 to 100psf depending on type of structure and direction
Allowable Stress Design (ASD)	Allowable Stress Design (ASD)+(LFD)
Earthquake load: NO	Earthquake: Equivalent lateral force
No transient Loads	No transient Loads

AASHTO (1996)	LRFD (1998)
H+HS+Design lane+Design tandem	H+HS+Design lane+Design tandem+Design Truck
LL Uniformly distributed	3 loads applied individually
Impact factor only dependent on Length of the span and <0.3	Impact factor depends on many factors and it is from 015 to 0.75 for different limit states
From 18 to 100psf depending on type of structure and direction	Depends on direction, velocity, pressure
Allowable Stress (ASD)+(LRFD)	Load and Resistance Factors (LRFD)
Earthquake: Equivalent lateral force+Response spectra	Earthquake elastic response coeff+equivalent weight+R response modification factor
No transient Loads	Transient loads: BR+CR+CT+VC

3. Load combination and Load distribution (longitudinal and transverse)

STIFFNESS AND FLEXIBILITY:

In an attempt to more accurately model the ***distribution of loads***, there are developed the LRFD distribution factors, which consider *span length* and *girder stiffness* as well as *girder spacing*. It is difficult to predict how the use of LRFD distribution factors will affect the final design moments and shears, in view of the relatively inconsistent results produced by the ASD distribution factors. If the remainder of the design specification remained unchanged, the significantly increased vehicular live loads would likely lead to *stronger and stiffer structures.* However, these changes in the load approximations are accompanied by *new, strength-based design procedures*. Such strength-based procedures have often led to *more flexible* structures in the past, so it is uncertain whether the increased loads will result in a comparable increase in bridge strength and stiffness. The revised load *distribution factors* further complicate the situation. A key issue in the application of the new specification is the flexibility of the resulting designs.

For a number of years, it has been observed that some steel-concrete composite bridges that have been designed according to AASHTO ASD standards have a tendency to display excessive flexibility. While the term "excessive flexibility" is somewhat subjective, and is often based upon the *perceptions passengers* in autos on the bridges, there may be more serious consequences for the bridge. Some bridges that have been observed to be too flexible also appear to have exhibited relatively *rapid deterioration of wearing surfaces*, and higher than expected *maintenance costs*. Whether there is any direct relation between the perceived flexibility of the bridge and the apparently higher rates of deck deterioration is not certain, but is consistent with observations that have been made over a number of years, by numerous engineers.

A summary paper (ASCE, 1958) indicated that, based upon available data at that time, bridge deflections were not considered to be a serious problem, except insofar as potential passenger/pedestrian discomfort were concerned. In recent years, most of the work related to deflection control has focused upon the passenger/pedestrian discomfort question. Little work on any *relationship* between *durability* and bridge *flexibility* appears to have been done since that time, although one of the recommendations of ASCE (1958) was specifically directed toward addressing this question. However, in the intervening years, there have been significant changes in the design of bridges. The development of LFD in AASHTO 1996 and more recently in AASHTO1998 LRFD design have lead to somewhat lighter-weight and potentially more flexible structures.

Specifically, the loadings upon which highway bridge design has been based for a number of years have been in existence since at least AASHTO1969. In the intervening years, there has been a significant observed increase in average daily truck traffic (ADTT) on many roads, and a significant increase in the observed weights of many trucks. While load limits are in place to ostensibly prevent bridge stresses from exceeding design stresses. Therefore, it is likely that many bridges on heavily traveled routes are subjected to *more cycles of high stress*, and quite possibly to higher stresses than the ASD HS-20 loading is intended to represent. The correct value of dynamic load allowances; the *impact factors*.

In addition to the presence of increased vehicular loadings, trends in bridge design over the last 50 years have tended toward more flexible bridge structures, so the L/800 limit for the design with the AASHTO1969, which has also been existence for many years, may in fact need to be modified. The original L/800 limit appears to have been based upon *non-composite bridges*, and the calculated deflections for such bridges were *based upon the girder stiffness only*, even though the actual bridges displayed considerable composite action in the field.

More recent design practice has explicitly included composite behavior in the design stage, and the stiffness estimates have been based upon the composite section. Therefore, application of the L/800 deflection limit to composite bridges may tend to permit significantly greater flexibility than would have been allowed in previous “non-composite” bridges that actually displayed significant composite action (ASCE, 1958).

Additional trends tending to contribute to greater flexibility include use of higher strength steels, which permit the use of smaller sections. Therefore, it is important to determine just how important the optional deflection provisions are, and whether more stringent guidelines should be followed in some circumstances.

This is particularly important in view of the observed flexibility of many composite steel girder-slab bridges, and the apparent tendency of some composite steel girder-slab bridges toward premature deck deterioration. It is the purpose of this study to evaluate the significance of those changes.

LOAD COMBINATIONS:

The LRFD (1998) Specifications use *limit states* instead of *load combinations* like in the previous AASHTO (1996) Specifications.

In the AASHTO (1996) several Group loading combinations for Service Load Design and Load Factor Design (LFD) are defined in function of particular sites or types of structure. For each Group N, the *group combination* is expressed by the following formula:

$$Group(N) = \gamma[\beta_D \cdot D + \beta_L(L+I) + \beta_C \cdot CF + \beta_E \cdot E + \ldots.]$$

also reported on the 3.22.1 of the code, with the list of all the loads
D is the dead load
L is the live load
I is the live load impact and so on for the other loads listed in the code
γ are the load factors pre-multiplier
β are the respective coefficients.

Whereas, in the LRFD Specifications different loading combinations are defined as *function of the Limit State* that it is required to achieve and satisfy. These limit states consist of the *service limit state* (limit state that requests provisions not directly related to strength or statistic but to the experience), the *strength limit state*. (limit state for which the structure is damaged but its main structural functions are still satisfied), the *fatigue limit state* (limit state that consider the presence of cracks due to applied loads but that prevents the fracture in the design life of the structure), and *extreme event limit state*. (limit state that assumes that the return period of loads which the bridge is subjected to is much greater than the design life of it, i. e.: earthquake).

In particular, in the code, the limit state is defined as "*a condition beyond which the bridge or component ceases to satisfy the provisions for which it was designed*." Thus, in example, Strength I is used for design at the Strength Limit State, when there is a normal vehicular use of the bridge and in the load combination do not appear the wind and other transient loads; instead the Extreme Event I associated with the Extreme Event Limit State includes the earthquake in the load combination; exc. All the non conformity to all the limit state is considered a failure (that does not compulsorily imply collapse).

These loading combinations are expressed by:

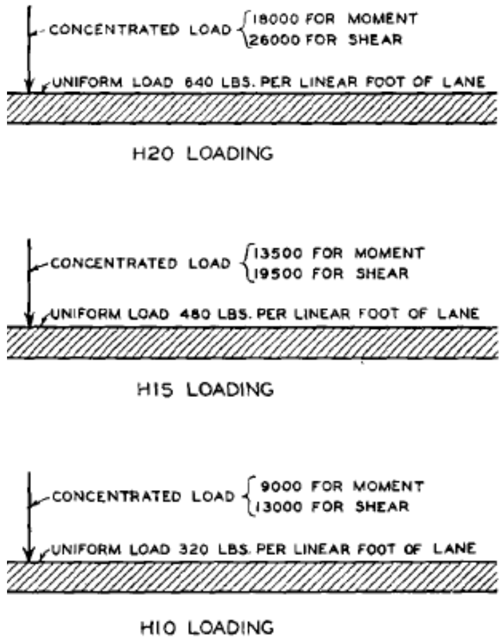
$$Q = \sum \eta_i \gamma_i Q_i \text{ ,}$$

where

η_i is the load modifier that takes in account redundancy, ductility and operational importance factors;

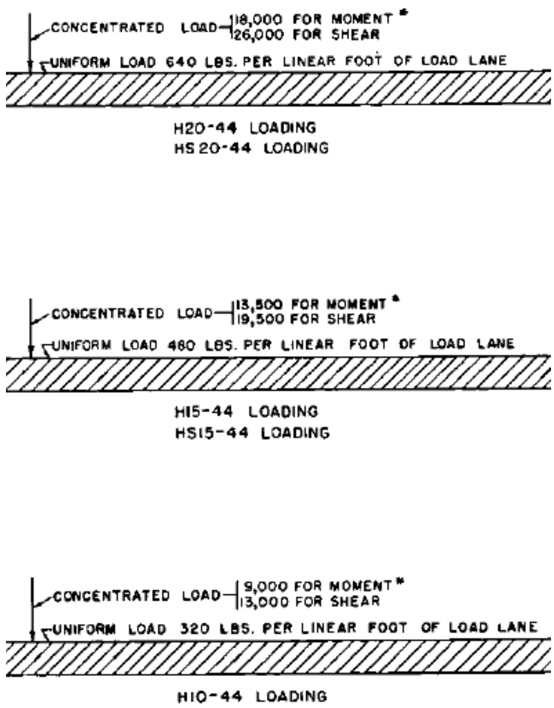
γ_i represents load factors that pre-multiply the force effects due to permanent and transient loads, Q_i.

1935



EQUIVALENT LOADING

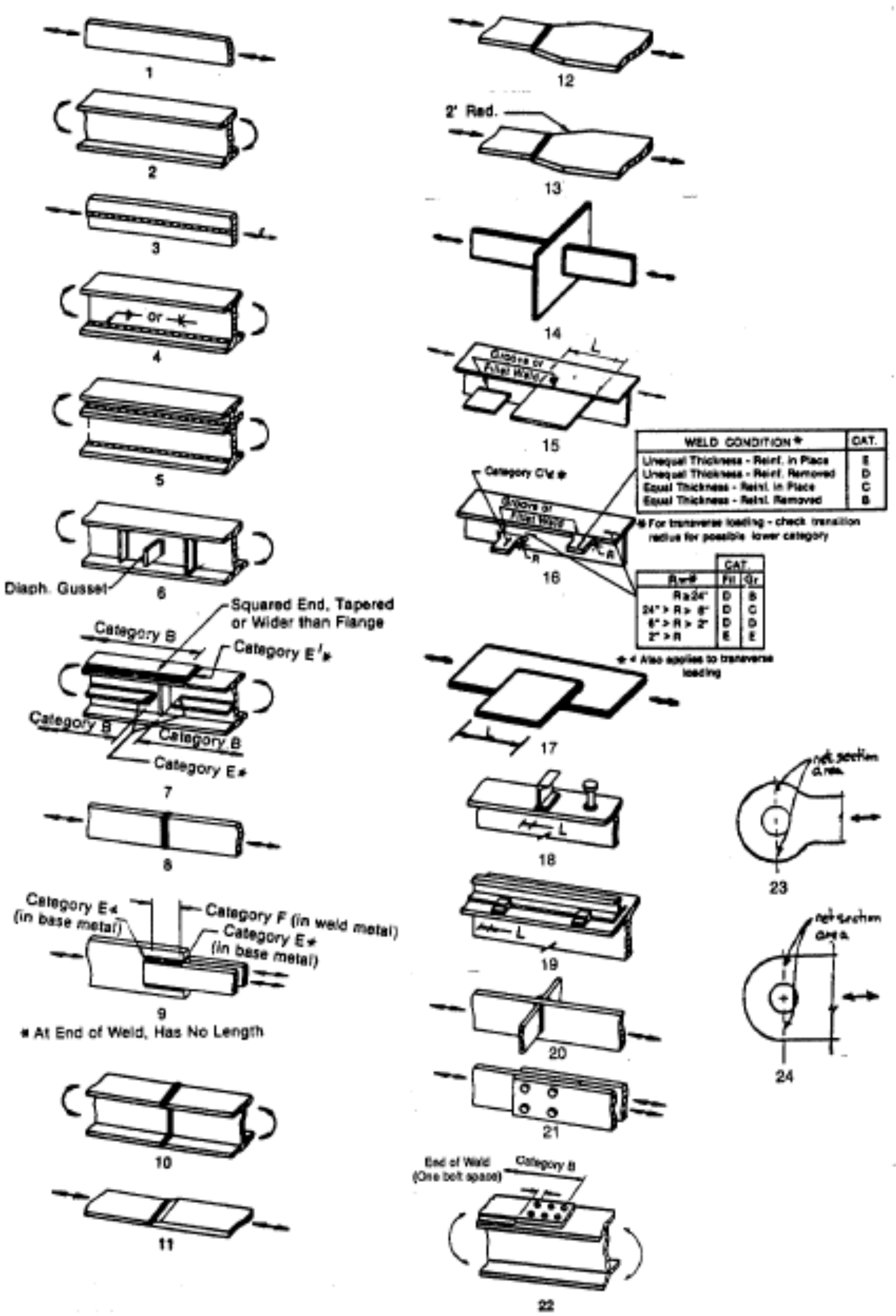
1969



H LANE AND HS LANE LOADINGS

Table of **fatigue** appear after failures in AASHTO 1992 and repeated also in the AASHTO 1998 after the failure of **Silver bridge, chain suspension bridge (Ohio River)** due to fatigue failure.

It appears the fatigue limit state and the follow table of joint to prevent failure:



EARTHQUAKE:

Past earthquakes in California have shown the vulnerability of some older structures, designed with *non-ductile design standard*s to *earthquake-induced force* sand *deformations*.

The ***Seismic Design Criteria*** (SDC) are an encyclopedia of new and currently practiced seismic design and analysis methodologies for the design of new bridges in California. The SDC adopts a *performance-based approach* specifying *minimum levels of structural system performance, component performance, analysis, and design practices for ordinary standard bridges*. Bridges with non-standard features or operational requirements above and beyond the ordinary standard bridge may require a greater degree of attention than specified by the SDC.

Many of the methodologies contained in the SDC have evolved from the seismic retrofit program. Some of the procedures are major departures from previous practice while others are slight modifications to current practice. The most significant change in design philosophy for new bridges is a shift from a ***force-based*** *assessment of seismic* ***demand*** to a ***displacement-based*** *assessment of* ***demand*** *and* ***capacity***.

The *force approach* was based on generating design level earthquake demands by reducing ultimate elastic response spectra forces by a *reduction factor*. The reduction factor was selected based on structure *geometry*, *anticipated ductility*, and *acceptable risk*.

The newly adopted *displacement approach* is based on *comparing the elastic displacement demand* to the *inelastic displacement capacity of the primary structural components* while insuring a minimum level of inelastic capacity at all potential plastic hinge locations. How bridges respond during earthquakes is complex. Insights into bridge behavior and methods for improving their performance are constantly being developed. Designers need to be conscious of emerging technology and research .

Bridges are categorized as either Important or Ordinary depending on the desired level of seismic performance. The Ordinary category is divided into two classifications Standard and Non-standard. A bridge's category and classification will determine its seismic performance level and which methods are used for estimating the seismic demands and structural capacities. The seismic design criteria for Important bridges and Ordinary Non-standard bridges shall be developed by the project design team on a case-by-case basis, and approved by OSD management.

4. Load distribution (longitudinal and transverse)

In the distribution of loads, there are several differences between the two Specifications, some of them can be considered minor differences, others are substantial.

In the AASHTO 1996 Specifications, in calculating *shear* and *bending moments* in longitudinal beams, the longitudinal distribution of the wheel load is usually not taken in account. While, the lateral distribution of wheel load, in calculating shear has to be that produced by assuming the flooring to act as a simple span between stringers and beams. For other loads in other positions on the span, the distribution for shear has to be determined using the method also specified for the moments.

The lateral distribution of wheel load in calculating live load bending moments for interior stringers and beams has to be considered by applying the fraction of a wheel load (both and rear) to the stringer. On the code a table reports these fractions of wheel loads that are expressed in function of the kind of floor and the number of traffic lanes. In the case of a span with concrete floor supported by 4 or more steel stringers, the fraction of the wheel load has to be:

For $6 < S \leq 14\,ft$

$$\frac{S}{4.0 + 0.25S}$$

For $S \leq 6\,ft$

$$\frac{S}{5.5}$$

where S is defined as the average stringer spacing in feet.

For multi-beams precast concrete bridges, conventional or prestressed, the longitudinal distribution of wheel load has always not to be considered; while the live load bending moment for each section has to be determined by applying to the beam the wheel load fraction to the beam (both front and rear):

$$S/D$$

S is the width of the precast member

D is a constant based on the properties of the bridge.

All these formulas for the distribution factors provide good results for bridges of typical geometry, but they are not so accurate when bridges have no common features and parameters, i. e., when relatively short or long span bridges are considered.

AASHTO LRFD Specifications, instead are more precise; consequently the formulas used for live load distribution factors are more complex and account for parameters such as *span length*, *girder spacing*, *cross-sectional properties of the bridge deck*, in particular, *flexural stiffness* and *torsional stiffness*. Furthermore, it is important to notice that LRFD code computes the live load distribution factor *per lane* rather than per wheel as in the AASHTO 1996 Specifications and that LRFD also includes *multiple presence factors* in the live load distribution factors. Here below are reported the distribution live loads per lane for bending moment in the interior beams:

For one design lane loaded

$$0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_S^3}\right)^{0.1}$$

For two or more design lanes loaded

$$0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_S^3}\right)^{0.1}$$

Where:
$3.5\ ft \le S \le 16\ ft$: spacing of primary members;
$20\ ft \le L \le 240\ ft$: span length;
$4.5\ in \le t_S \le 12\ in$: depth of concrete slab;
K_g: longitudinal stiffness parameter depending on the area of the cross-section and on the moment of inertia of the beam (not all these parameters are considered by the AASHTO 1996).

Regarding the transverse distribution of the wheel load in calculating the bending moments in floor beams, the AASHTO 1996 does not consider it. Instead, if longitudinal stringers are omitted and the floor is supported directly on floor beams, the transverse beams have be designed for loads determined by applying to the stringer a fraction of wheel load, that is a function of S, spacing of floor beams in feet and changes its expression according to the kind of floor. In the code is reported a table for those distribution of wheel load in transverse beams.

In the LRFD Specifications, for flexural moments and shear, if the deck is supported directly by floor-beams, the transverse floor-beams can be designed for loads determined in accordance with a table reported in the code that consists of the fraction of wheel loads to each floor beam in function of the type of deck and of S. It is very similar to the same table reported in the old code (1996), above mentioned. Instead, in LRFD, the live load per lane for shear in interior beams can be determined by applying the lane fractions specified in another table in the code, in functions of the type of superstructure, cross-section, number of design lane loaded and range of applicability.

In the following list of bridge failure it is written what was the cause. I would like to point out that in all bridge failures there are not only one cause, but several cause are responsible for failures; very often nothing would happened if only one hazardous event occur. Obviously it is possible to prioritize these failures and very often determine what is the trigger of failure.

The most common list of cause can be classified as

- Failure of foundation
- Failure due to unusual effect or impact
- Collapse during construction or dismantling
- Failure during testing
- Insufficient load-bearing capacity without other recognizable cause

Where there could be mistake in design, detailing and erection, flaws in maitenance, and material and natural disaster.

In the following list of bridge failures, the causes are selected fro the previous classification after a forensic investigation. These can provide information and material to affect eventual changes in design and/or construction practices, codes, standards, oversight and regulatory procedures.

3. LIST OF BRIDGE FAILURES

Cable suspension bridge near Wheeling (Ohio River), 1854

Cable suspension bridge Lewiston-Queenston (Niagara)

Ashtabula (cast iron) bridge, Ohio, 1876

Farmington Bridge Failure, Connecticut,1878

Truss bridge near St. Charles (Missouri River), 1879

(The Tay disaster between Dundee and Wormit (Scotland), 1879)

2-span truss bridge near Fish's Eddy, New York, 1886

Bridge over Wabash River crossing Toledo St. Louis and Kansas City Railway, Bluffton(Indiana),1886

The iron Ducannon bridge, Pennsylvania, 1886

Hilton bridge between Wilmington Columbia and Augusta railway (North Carolina), 1886

Pratt truss on the New York Ontario and Western railway, 1886

Iron Whipple truss over Petewawa River, Pembrook (Ontario) 1886

Bridge near Louisville, Nashville (Alabama River), 1887

Bussey bridge disaster near Forest Hill (Boston), 1887

Big Otter River, Northfolk and western Railway (Virginia), 1887

Small wooden bridge in North Chatsworth, Illinois, 1887

Staunton Bridge, Virginia, 1887

La Salle bridge, New York, 1889

St. George, Ontario, 1889

Bridge crossing the Knoxville Cumberland Gap and Luissville Railway over Flat Creek, 1889

The Pekin Peoria and Union Bridge, over the Illinois River (Illinois), 1890

San Bernardino Bridge, California, 1890

(Moenchenstein Disaster, 1891)

Bridge in Chicago, 1892

Covington Bridge, 1892

Denville, Illinois, 1893

Chester truss bridge, 1893

Louisville truss bridge, 1893

Two span of a street railway bridge in Saginaw, Michigan, 1894

Bedford Bridge, Ohio, 1896

Bridge at Point Ellic, Victoria (British Columbia)

Birningam Bridge over the Canhaba River, Alabama, 1896

Bridge near Spartanburg, South Carolina, 1897

Viaduct in Pottsville, Pennsylvania, 1897

Bridge over the St. Lawrence River near Cornwall, by the Ontario-New York Railway, 1898

Highway Bridge in Shelby, Ohio, 1898

Porter's Draw timber railroad bridge, north of Pueblo, Colorado, 1904

3 arches Auburn bridge, California, 1911

Glen Loch bridge, Pennsylvania, 1912

Coos-Bay-bridge Oregon, 1924

Ohio Falls truss bridge, 1927

Poughkeepsie suspension bridge, 1927

4-span beam and slab bridge (Anacostia River), 1933

Steel truss bridge near Manassas, Virginia, 1937

Truss bridge near Pagosa Springs in Colorado, 1937

Whiteson Bridge near Minnville, Oregon (North Yamhill River), 1937

Bridge near the Niagara Falls (Niagara River), 1938

Plate girder Gerber hinge bridge near New York, 1939

Tacoma Narrows suspension bridge, 1940

Two U-section bridges south of Le Mars, Iowa (Floyd River), 1941

2-span truss bridge over Mississippi in Chester, Illinois, 1944

Swing bridge in Boston-Charlestown, Massachusetts, 1945

John Grace-Memorial Bridge (Cooper River), South Carolina, 1946

Bridge near Fresno, California (King's Slough River), 1947

Rockport-Bridge, Maine (Goose River), 1947

Hinton truss bridge, West Virginia, 1949

Elbow Grade Bridge, Willamette National Forest, timber truss, 1950

Sullivan Square motorway bridge, Boston, 1952

Eric bridge, Cleveland, Ohio, 1956

Interstate 29 West Bridge, Sioux City, Iowa, 1962

Lake Pontchartrain bridge (Lake Pont), 1964

Bridge near Charleston, South Carolina (Cooper River), 1965

Silver bridge, chain suspension bridge (Ohio River), 1967

Bridge in Illinois (Kaslaski River), 1970

Buckman Bridge near Jacksonville, Florida, 1970

Chesapeake Bay Bridge, Annapolis, 1970

Motorway bridge, Junction Antelope Valley, 1971

Motorway bridge near Pasadena, California (Arroyo Seco River), 1972

Sidney-Lanier Bridge Brunswick, Georgia, 1972

Chesapeake Bay Bridge, Annapolis, 1972

Lake Pontchartrain bridge (Lake Pont), 1974

3-span bridge in Lafayette Street, St-Paul, Minnesota, 1975

21-span, Pass Manchac Bridge, Louisiana, 1976

Fulton Yates Bridge near Henderson, Kentucky, 1976

Benjamin Harrison Memorial Bridge near Hopewell, Virginia, 1977

Bridge over Passiac River, Union Avenue, 1977

Interstate 17 Bridge, Black Canyon, Arizona, 1978

Southern Pacific Railroad Bridge (Berwick Bay), Louisiana, 1978

K&I Railroad Bridge, Louisville, Jefferson County, Kentucky, 1979

Alabama Rail Bridge, Alabama, 1979

Southern Rail Bridge, Indiana, 1979

Bridge over the Hood canal, Washington, 1979

Interstate 10 Bridge, Phoenix, Arizona, 1979

Concrete 5-span box girder bridge near Rockford, 1979

Sunshine Skyway Bridge near St. Petersburg, Florida, 1980

Truss bridge in Trenton, Wisconsin (Milwaukee River), 1980

Multiple span box girder bridge in East Chicago, Indianapolis, 1982

Prestressed concrete precast box girder bridge, Saginaw, 1982

Syracuse bridge, New York, 1982

Connecticut Turnpike Bridge near Greenwich (Mianus River), 1983

Walnut street viaduct over Interstate 20 in Denver, Colorado, 1985

Bridge in El Paso, Texas, 1987

Schoharie Bridge (New York), 1987

Motorway bridge near Seattle, 1988

Box girder bridge in Los Angeles, 1989

Bridge in Baltimore, 1989

Cypress Freeway, Oakland, California, 1989

Section of East span of San Francisco Oakland Bay Bridge, California, 1989

Truss bridge in Shepherdsville, Kentucky, 1989

Herbert C. Bonner Bridge, North Carolina, 1990

Motorwaybridge, jonction Antelope Valley, 1992

Truss bridge in Concord, New Hampshire, 1993

Truss bridge near Mobile, Alabama, 1993

Interstate 5 Bridge in Los Angeles, California, 1994

3-span 3-girder composite bridge near Clifton (Tennessee River), 1995

Twin bridges, Interstate 5 (Arroyo Pasajero River), Coalinga, California, 1995

Walnut Street Bridge in Harrisburg, Pennsylvania (Susquehanna River), 1996

Bridge near Covington, Tennessee (Hatchie River), 1999

Bridge over motorway in Concord, North Carolina, 2000

Queen Isabella Causeway, Texas, 2001

Historic Tewksbury Township pony truss bridge, Hunterdon County, New Jersey, 2001

Interstate 40 Bridge, Oklahoma (Webber Falls), 2002

Marcy bridge (Utica-Rome Expressway project), 2002

Turkey Creek Bridge, Sharon Springs, Kansas, 2002

Highway 14 overpass, 60 miles south of Dallas, Texas (over Interstate 45), 2002

1900 built Kinzua Viaduct (north-central Pennsylvania), steel bridge, 2003

Imola Avenue Bridge, Napa, California, 2003

Interstate 95 Bridge in Bridgeport, Connecticut, 2004

West Grove Bridge in Silver Lake, Kansas, 2004

Interstate 20 Bridge near Pecos, Texas (Salt Draw River), 2004

Lee Roy Selmon Expressway, Tampa Bay, Florida, 2004

Bridge near Pawnee City, Nebraska, 2004

Shannon Hills Drive Bridge, Arkansas, 2004

Interstate 70 Bridge in Denver, Colorado, 2004

Interstate 10 Bridge, Escambia Bay, Florida, 2004

McCormick County bridge east of Mount Carmel (Little River), South Carolina, 2004

Bridge northwest of Norcatur (Sappa Creek), Kansas, 2004

Rural bridge near Shelby, North Carolina (Beaver Dam creek), 2004

Laurel Mall Pedestrian Bridge between the parking and shopping areas, 2005

Wooden bridge in Pico Rivera (California) spanning the Rio Hondo flood-control channel, 2005

Lake View Drive Bridge, Interstate 70 in Washington County (Pennsylvania), 2005

Eight-lane, 1,950-feet-long Interstate 35-West Bridge in Minneapolis (Mississippi River), 2007

4. COLLECTION DATA

Cable suspension bridge near Wheeling (Ohio River)

Year	1854
Type	Road
Country	United States
Cause	Natural hazard (Storm)
Details	Storm
Fatalities	0
Injuries	0
Collapse	complete
Phase	in service
References	SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

It was the largest suspension bridge in the world from its completion in1849, until the Queenston-Lewiston Bridge was opened in 1851. It was the first to span the Ohio River and it was designed by Charles Ellet Jr., who also worked on the Niagara Falls Suspension Bridge, failed ten years later. The main span is 1,010 feet (310 m) from tower to tower. The east tower is 153.5 feet (46.8 m) above the low-water level of the river, or 82 feet (25 m) from the base of the masonry. The west tower is 132.75 feet (40.46 m) above low water, with 69 feet (21 m) of masonry. On May 17, 1854 a strong windstorm destroyed the deck of the bridge through torsional movement and vertical undulations that rose almost as high as the towers

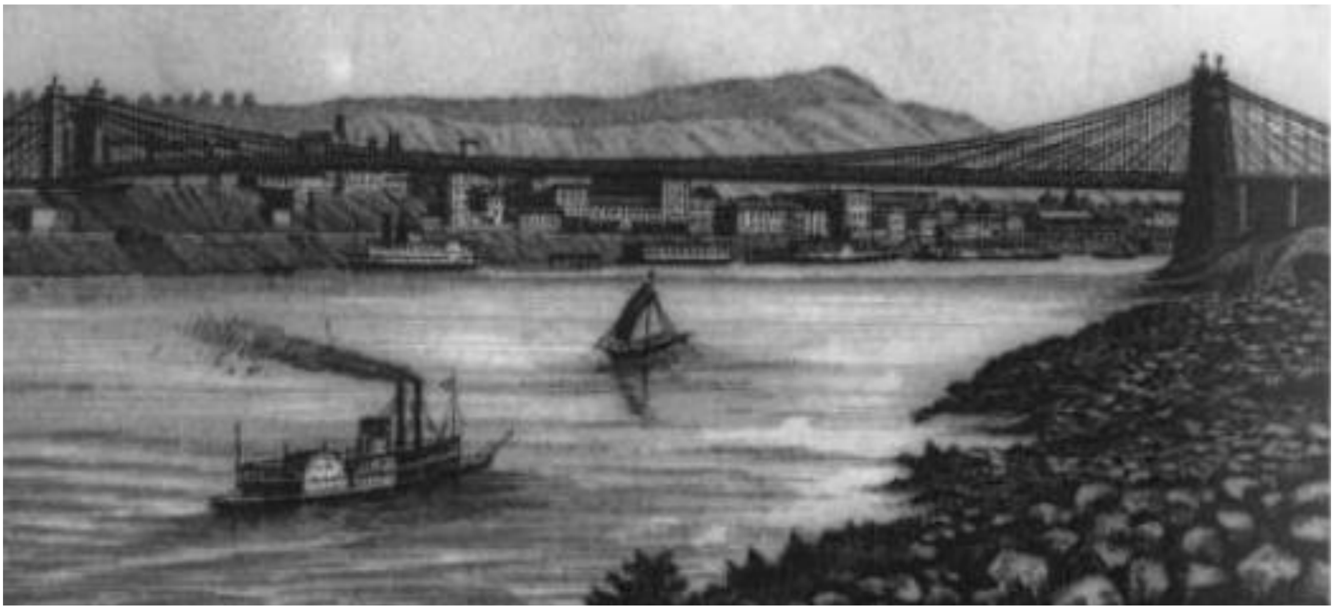


Figure 1

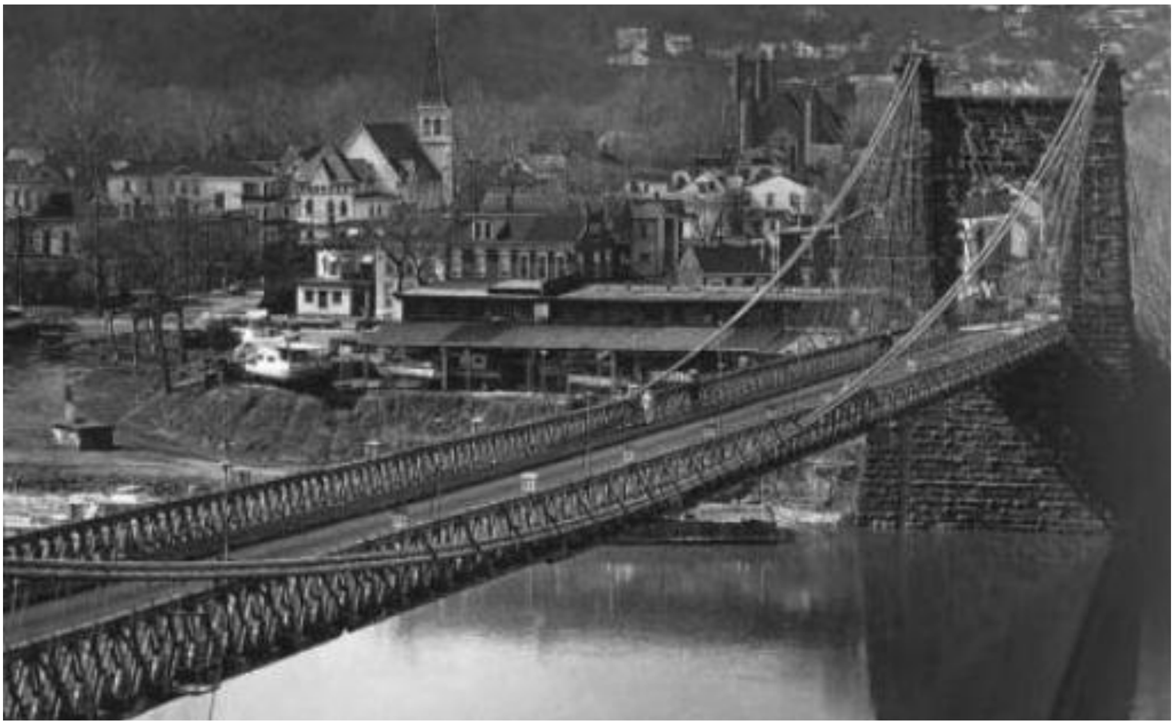


Figure 2

Cable suspension bridge Lewiston-Queenston (Niagara)

Year 1864

Type Road

Country United States

Cause Natural hazard (Storm)

Details Storm

Fatalities 0

Injuries 0

Collapse complete

Phase in service

References SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

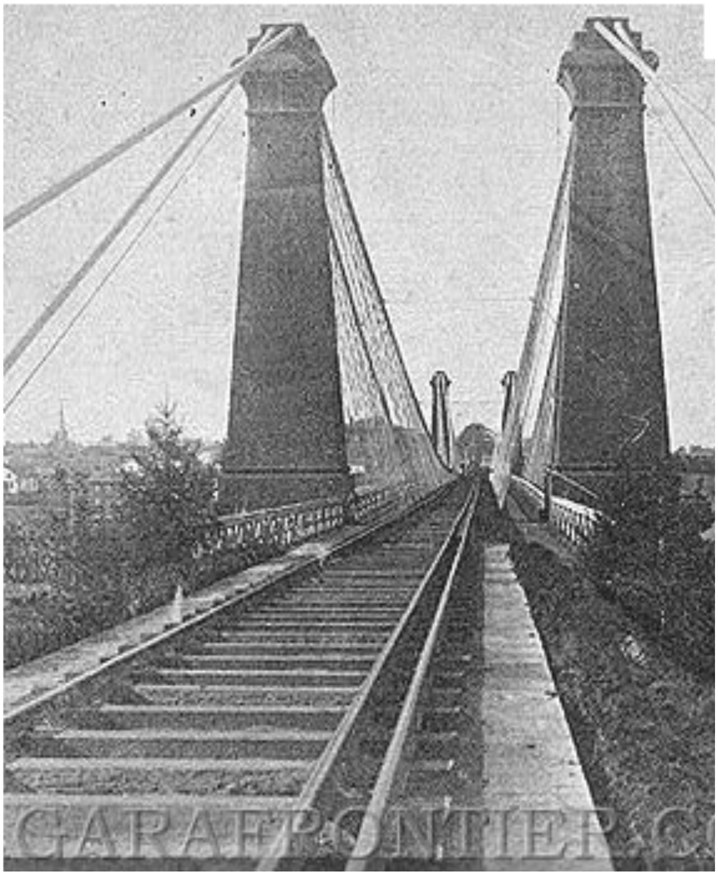


Figure 3

Ashtabula (cast iron) bridge, Ohio

Year 1876

Type Rail

Country United States

Cause Overloading

Details Very heavy train, snow storm, fatigue not excluded

Fatalities 80

Injuries 0

Collapse complete

Phase in service

References SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.
BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002.
http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php
http://www.englib.cornell.edu/ice/lists/historytechnology/successfailures.html
http://www.iti.nwu.edu/links/bridges/disasters.html

This bridge was a two truss, double track, parallel chord, deck bridge with the web members arranged as in the Howe truss (see reference http://www.past-inc.org), but without the vertical end post. There were 14 panels of 11ft each with a height of 19ft 9in. The distance between centers of the trusses was 16ft 6in, and the width of each truss was 34-in.

The compression members of the bridge have no lateral connections with each other, thus acting as separate long slender columns that cause weaknesses for buckling.

Another weak point was in the joints; Castiron joint blocks were used at all joints, and the top chord members simply butted against lugs of these castings. Moreover the details of the jounction were not designed in detail and therefore after the erection the beams were left in contact with the blocks only at one point, so they acted as columns with free ends.

Therefore the structure was deficient in many parts and the failure of one of these weak parts was the trigger.

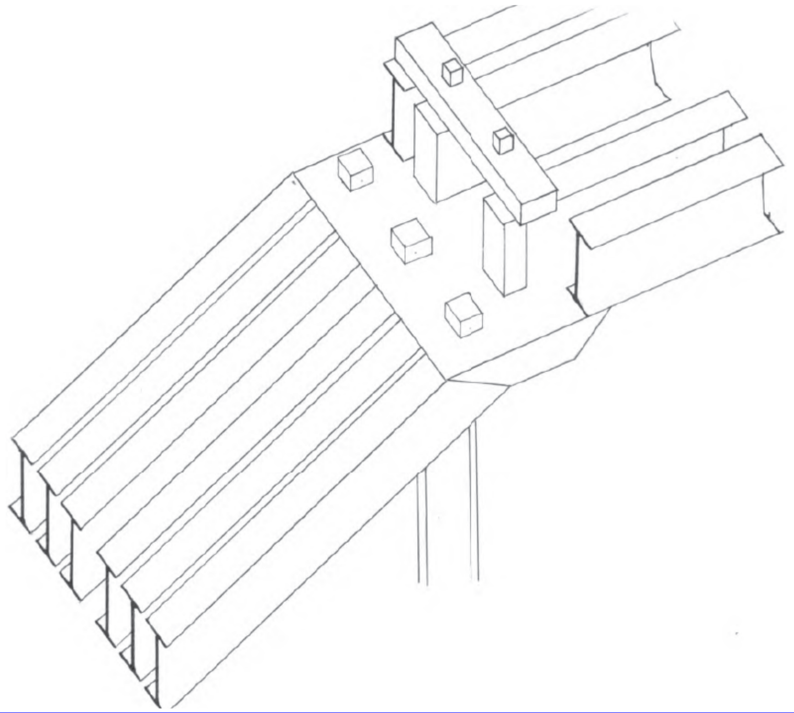


Figure 4: Joints acting like free hands

Figure 5

Farmington Bridge Failure, Connecticut,1878

Year	1878
Type	Rail (Combination of wood and steel)
Country	United States
Cause	Material quality
Details	Storm
Fatalities	17
Injuries	43
Collapse	complete
Phase	in service
References	SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

The bridge was a through span, 162 ft in the clear, of Howe type, the chords and diagonal of pine, and verticals wrought iron. The bridge failed under a double-headed ten-car Connecticut Western Railroad special train of the faithful, returning from a revival held in Hartford, crosses the Tariffville Bridge over the Farmington River near midnight, and the structure collapses. Both locomotives and the first four cars plunge into the ice-covered river, killing seventeen and injuring forty-three. Study on the material stress show that the members were all to be scant of material and besides this wooden member were considerably decayed. State inspections failed.

Truss bridge near St. Charles (Missouri River), Wreck

Year	1879
Type	Rail
Cause	Impact
Details	Derailed train impact on bridge
Fatalities	2
Injuries	0
Collapse	partial
Phase	in service
Reference	SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

This bridge was a span of the Howe type. It was cracked after the derailment of the bridge of a freight train; the floor beams were of wood, and the breaking of one of these floor beams caused the derailment cars being thrown against the trusses causing them to fall.

Figure 6

The Tay disaster between Dundee and Wormit (Scotland)

During a violent storm on the evening of 28 December 1879, the centre section of the bridge, known as the "High Girders", collapsed, taking with it a train that was running on its single track. All seventy-five people on the train were killed.

Year 1879

Type Rail (Continuous girder bridge, wrought iron framework on cast iron columns, railway bridge)

Country United Kingdom (Scottland)

Cause Design

Details Overturning piers

Fatalities 75

Injuries 0

Collapse partial

Phase in service

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000
http://en.wikipedia.org

Many faults in design, materials, and processes that had contributed to the failure.

Bouch had been advised that calculating wind loads was unnecessary for girders shorter than 200 feet (61 m), and had not followed this up for his new design with longer girders. The section in the middle of the bridge, where the rail ran inside high girders (through trusses), rather than on top of lower ones (deck trusses), to allow a sea lane below high enough for the masts of ships, was potentially top heavy and very vulnerable to high winds. The piers consisted of four castiron columns 15in. In diameter, and two columns 18in in diameter, in the form of cylinders, allo ne and a half inches in thickness, and filled with portland cement concrete. These seven pieces were put together by horizontal and diagonal braces.The wind pressure requie to overturn the structure, with a train on it was not over the 50lbs per sq. ft. The piers being light lacce the weight to resist overturning.

Figure 7

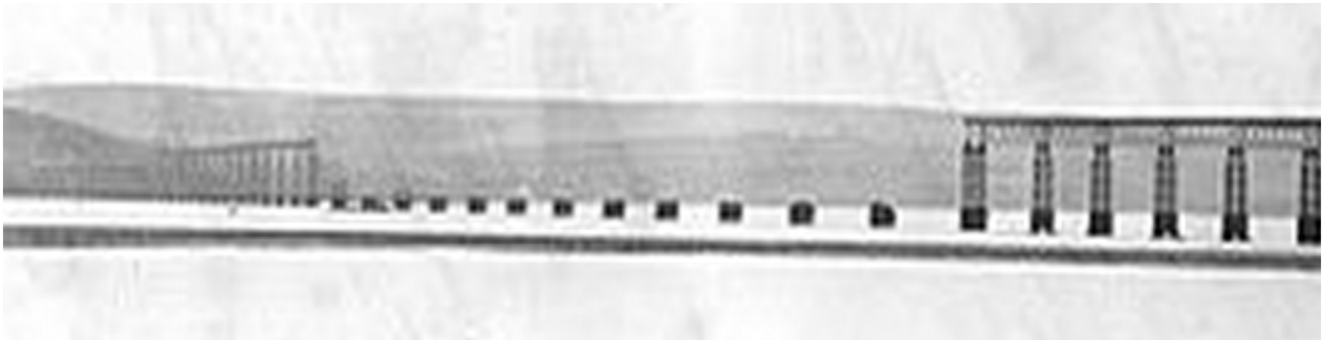


Figure 8

2-span truss bridge near Fish's Eddy, New York

Year 1886

Type Rail

Country United States

Cause Impact

Details Derailed train impact on bridge

Fatalities 0

Injuries 0

Collapse complete

Phase in service

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000

Bridge over Wabash River crossing Toledo St. Louis and Kansas City Railway, Bluffton(Indiana),1886

Year 1886

Type Rail (combination of wood and steel)

Cause Impact

Details Center pier washed out

Fatalities 2

Injuries 2

Collapse Total

Phase in service

Reference University of Wisconsin--Madison. College of Engineering

Truss bridge near Ducannon, (Pennsylvania)

Year 1886

Type Rail

Cause Impact

Details Center pier washed out

Fatalities 1

Injuries 6

Collapse Total

Phase in service

Reference University of Wisconsin--Madison. College of Engineering

This was an old iron Pratt truss bridge previously been strengthened by castiron arches. Afterwards a pier was built under the center of the span and the arch removed. A freshet washed out this pier and when the freight train cross the river it failed.

Iron bridge in Hilton, (North Carolina)

Year 1886

Type Rail and car

Country United States

Cause Impact

Details A dereiled car strinked the end post

Fatalities 0

Injuries 0

Collapse Partial

Phase in service

Reference University of Wisconsin--Madison. College of Engineering

Pratt truss on the New York Ontario and Western railway

Year 1886

Type Rail and car

Country United States

Cause Impact

Details Truss knocked down by dereiled cars

Fatalities 4

Injuries several

Collapse Partial

Phase in service

Reference University of Wisconsin--Madison. College of Engineering

Iron Whipple truss over Petewawa River, Pembrook (Ontario)

Year 1886

Type Rail

Country United States

Cause Impact

Details Steam sholvel too high struck the portal

Fatalities 2

Injuries 20

Collapse Partial

Phase in service

Reference University of Wisconsin--Madison. College of Engineering

A through iron whipple truss on the Canadian Pacific Rail-way was knocked down; a steam shovel was too high and in going through struck the portal, causing the end post to raise and wreck the bridge

Bridge near Louisville, Nashville (Alabama River)

Year	1887
Type	Rail
Cause	Natural hazard (Debris in water)
Details	Wood in water destroys bridge
Fatalities	0
Injuries	0
Collapse	complete
Phase	in service
Period	before 1900
Continent	North America
Reference	SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Bussey bridge near Forest Hill (Boston)

Year 1887

Type Road

Country United States

Cause Design error/poor construction

Details Error in design and construction

Fatalities 26

Injuries 115

Phase in service

Period before 1900

Continent North America

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

This is one of the most disastrous failure that has evere been in terms of life lost and injuried. The bridge was 104ft span, built on a skew, over a highway. A morning commuter train, inbound to Boston, was passing over the "Bussey Bridge", a Howe truss, at South Street collapsed, sending several cars crashing to the street below. Twenty-four commuters were killed and another 125 were seriously injured. The breaking of an iron hanger, that support the floor beams to the upper chord of the deck truss, due to the propagation of an old crack.

The design was very poor design because the load acts to one side of the resisting force, producing bending moment instead of pure traction. The breacking of the hanger is caused by a locomotive that support an engine.

Small wooden bridge in North Chatsworth, Illinois

Year	1887
Type	Rail
Country	United States
Cause	Human error
Details	Bridge caught fire after weeds had been burnt along the track earlier. A six coach Niagara Falls Special train unable to stop in time.
Fatalities	82
Injuries	0
Collapse	complete
Phase	in service
Period	before 1900
Continent	North America
References	http://www.basedn.freeserve.co.uk/bridge.htm http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php

Big Otter River, Northfolk and western Railway (Virginia), 1887

Year	1887
Type	Rail
Country	United States
Cause	Human error
Details	Bridge caught fire after weeds had been burnt along the track earlier. A six coach Niagara Falls Special train unable to stop in time.
Fatalities	82
Injuries	0
Collapse	complete
Phase	in service
Period	before 1900
Continent	North America
References	University of Wisconsin--Madison. College of Engineering

The bridge was a timber Howe truss. Later the timber truss was replaced by a Fink truss of iron, The third span was completed with the exception of the sway bracing, where in place of this, diagonal wooden planks had been inserted. In this condition several train passed the bridge, and the heavy coal train caused the failure.

The failure might have been caused by the breaking of I bars, as two were found in the wrecks showing old fractures in welds, that might have been due to the stresses caused by lateral deflection.

Staunton Bridge, Virginia

Year 1887

Type Rail

Country United States

Cause Human error

Details Weakening of steel member by overheating (as timber bridge is replaced by steel bridge)

Fatalities 0

Injuries 0

Collapse partial

Phase construction

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

La Salle bridge, New York, 1889

Year 1989

Type Rail

Country United States

Cause Impact

Fatalities 0

Injuries 0

Collapse total

Phase In service

Reference University of Wisconsin--Madison. College of Engineering.

A derailed car "swelling" out against the 100' Pratt truss that was good designed; It fell while a freight train was passing over it.

St. George, Ontario, 1889

Year	1989
Type	Car
Country	United States
Cause	Poor constrction details
Fatalities	13
Injuries	29
Collapse	partial
Phase	In service
Reference	University of Wisconsin--Madison. College of Engineering.

This was a eight span of 50'bridge of lattice girders. The girder was designed properly and all the requirements are respected; the weak part was the floor of the bridge, consisting in 8'' by 12'' ties spaced 10''apart and resisting directly upon the girders. There were no rerailing device along the end of the tie, so that just before the bridge was reached one of the driver tires broke and rolled off to one side of the road. If the rerailing devices had been used on the bridge and the floor system had been well designed, no serious results would have occurred.

Bridge crossing the Knoxville Cumberland Gap and Luissville Railway over Flat Creek, 1889

Year	1989
Type	Car (wood bridge)
Country	United States
Cause	Impact/ Dereilment
Fatalities	10
Injuries	19
Collapse	total
Phase	In service
Reference	University of Wisconsin--Madison. College of Engineering.

There was a derailment at about 200’ from the trestle; the derailed car kept on the road bed until the trestle was reached, breaking a beam ends; this beam encountered the ends of the loose ties of the trestle, carrying them forward and bunching them together. The bunching left a gap in the deck that create a truck accident.

The accident could have been be averted had rerailing device been used and had the floor been strong. The floor was very poor, the ties being spaced 4’ apart.

The Pekin Peoria and Union Bridge, over the Illinois River (Illinois), 1890

Year	1890
Type	Rail
Country	United States
Cause	Not found
Details	Failure under 86 tons engine
Fatalities	0
Injuries	0
Collapse	complete
Phase	in service
Reference	University of Wisconsin--Madison. College of Engineering.

San Bernardino Bridge, California, 1890

Year	1890
Type	Rail/ 300' suspension bridge
Country	United States
Cause	Material strength
Details	8 teams of mules and two wagons for an amount of 120 lbd (designed for 1500 lbs per linear ft)
Fatalities	0
Injuries	0
Collapse	complete
Phase	During tests
Reference	University of Wisconsin--Madison. College of Engineering.

Bridge in Chicago

Year 1892

Type Rail

Country United States

Cause Ship Impact

Details A steamer hit the bridge/ error of ship captain

Fatalities 0

Injuries 0

Collapse complete

Phase in service

Continent North America

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Figure 9

Covington Bridge

Year 1892

Type Road

Country United States

Cause Design error (probably)

Details Cable failure

Fatalities 26

Injuries 12

Collapse complete

Phase construction

Period before 1900

Continent North America

Reference University of Wisconsin--Madison. College of Engineering
SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

That was a highway bridge with several short span, with the longest one of 300', which failed under construction. The false-work are strong enough and the construction procedures were the same of many other works with the same type of methodology.

The most probable cause was the poor construction phase design to take into account the settlement of the false-work that were supported on soft soil.

Denville, Illinois

Year 1893

Type Rail

Cause Impact

Details Freight train broke in two; the bridge was shoved forward allowing the rear endto drop off its pier

Fatalities -

Injuries -

Collapse partial

Phase In phase

Chester truss bridge

Year 1893

Type Rail

Country United States

Cause Human error

Details Train enters bridge on which some load-bearing elements were removed

Fatalities 17

Injuries 32

Collapse complete

Phase Construction/repair

The wrecked bridge was two 110’. The bridge was undergoing repairs, consisted of putting on more cover plates to the top chords, to meet the demands of an increase of weight of rolling stock. They put new elements to carry the load, but since the old members are carrying the dead loads, the new plates cannot be worked up to their available strength without overloading the old part. The way to obtain equal stress is to jack up the truss so as to relieve the dead load in the members being strengthened.

Louisville truss bridge

Year	1893
Country	United States
Cause	Human error
Details	Parts of scaffolding removed before bracing of bridge put in place, strong winds
Fatalities	22
Injuries	0
Collapse	partial
Phase	construction
Period	before 1900
Continent	North America
Reference	University of Wisconsin--Madison. College of Engineering SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

The bridge is two 338' spans, three 546.5' spans and one 208' span; thirty-seven individuals died during its construction. The first twelve died while working on a pier foundation when a caisson that was supposed to hold back the river water flooded, drowning the workers. Another four men died a few months after that when a wooden beam broke while working on a different pier caisson.

The Big Four Bridge had one of the biggest bridge disasters in the United States, occurring on December 15, 1893 when a construction crane was dislodged by a severe wind, causing the false-work support of a truss to be damaged and the truss—with forty-one workers on it—fell into the Ohio River. Twenty of the workers survived, but twenty-one died. Hours later, a span next to the damaged span also fell into the river, but was abandoned at the time, causing no injuries. The failure was caused by the wind that overturn the traveler, putting all the weight in the crane corner. The contractor stated that the wind picked up the truss bodily and dropped it into the river. However it seems improbable considering that the truss weighted only 2M lb and the wind was 30m/h. So it was developed that not all the lateral bracing ware in place, and probably it accounts for the failure.

Figure 10

Figure 11

Two span of a street railway bridge in Saginaw, Michigan

Year	1894
Type	Street Railway
Country	United States
Cause	Wind Storm + poor construction design
Details	Not enough stiffened
Fatalities	0
Injuries	0
Collapse	partial
Phase	Construction
Continent	North America
Reference	University of Wisconsin--Madison. College of Engineering SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

The bridge was two span; one was a combination of Warren truss of 160' span, and the other a swing bridge combination Howe truss of 160' span. The failure of the first span was due to the weakness of the structure, the wind and lateral bracing failing first and allowing the trusses to fall sideway. The bridge was constructed in a different way respect to the one that the engineer thought, substituting the iron with the wood and without designing calculation on it.

The pressure due to overturning effect of the wind caused the turn table to fail, allowing the bridge to slip off sideway.

In other parts the rived were spaced too far in the end post that the cover plates are bulged up between the rivets.

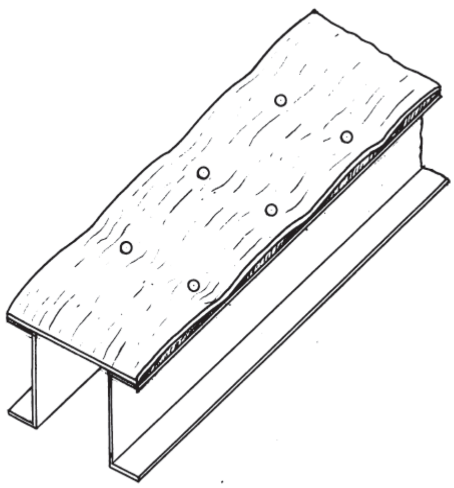


Figure 12

Bedford Bridge, Ohio

Year 1896

Type Rail

Country United States

Cause Limited knowledge

Details Not enough stiffened

Fatalities 0

Injuries 0

Collapse complete

Phase in service

Continent North America

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Figure 13

Bridge at Point Ellic, Victoria (British Columbia)

Year 1896

Type Road/ combination of wood and steel

Country United States

Cause Limited knowledge

Details Not enough stiffened

Fatalities 70

Injuries 200

Collapse complete

Phase in service

Reference University of Wisconsin--Madison. College of Engineering
SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

The bridge was 120' deck span of Pratt combination trusses, two 120' deck span of Whipple combination trusses, and short trestle approach and it was designed first to have a wagon bridge but then it was changed during the buildup extending the gauge, using flat strap rails and spiking them to the floor planks, no extra stringers being used.

In order to develop the bending moment on the floor beams the track was laid on one side, but this increase the shear on that side and so weakened the floor beams.

After an accident, the old floor was substitute but some of them were kept in place; it was a failure of one of these floor beams that caused the wreck. It sheared off close up to the end where the wood had decayed.

Birningam Bridge over the Canhaba River, Alabama

Year 1896

Type Rail

Country United States

Cause Loosening or removal of a rail by train wrekers

Fatalities -

Injuries -

Collapse partial

Phase in service

Reference University of Wisconsin--Madison. College of Engineering
SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

It was a bridge with several short span from 30' to 60' and the central one of 150'. The central span, two of sixty and one of thirty feet failed. The wreck have been caused by the loosening or removal of a rail by train wreckers.

Bridge near Spartanburg, South Carolina

Year 1897

Type Rail

Country United States

Cause Impact

Details Derailed train impact on bridge

Fatalities 0

Injuries 0

Collapse complete

Phase in service

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000

Viaduct in Pottsville, Pennsylvania

Year 1897

Type Viaduct

Country United States

Cause Pour design

Details Failure of one of the abutments

Fatalities 0

Injuries 0

Collapse complete

Phase in service

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000

The wing-walls, built against the side of a bluffer were not heavy enough to resist the swelling action of the wet bank

Bridge over the St. Lawrence River near Cornwall, by the Ontario-New York Railway

Year 1898

Type Viaduct

Country United States

Cause Pour design

Details Failure of one of the abutments

Fatalities 15

Injuries 16

Collapse Construction

Phase Not in service

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000

It was erected as a cantilever bridge in one span, and in the other two as a composite bridge of Pratt truss span of 368' each. The two further span were already completed and the one next to the shore practically completed where a worker tearing down the false-work and the two span fell.

The conclusion were that the pier must have fallen frst, but the suddenness of the failure led many to believe other-wise, as masonry always gives warnings before failure. But it is possible that the piers was undermined and so it might topped over suddenly.

Porter's Draw timber railroad bridge, north of Pueblo, Colorado

Year	1904
Type	Rail
Country	United States
Cause	Natural hazard (Flooding)
Details	30 feet of floodwater that swept through the normally dry channel, washing out the county bridge. The bridge floated downstream and severely damaged the wooden railroad bridge. Train caused weakened bridge to collapse
Fatalities	97
Injuries	0
Collapse	complete
Phase	in service
Reference	http://www.denverpost.com/Stories/0,1413,36~53~2321126,00.html

3 arches Auburn bridge, California

Year 1911

Country United States

Cause Design error

Details Scaffolding collapses under weight of fresh concrete

Fatalities 3

Injuries 16

Collapse complete

Phase construction

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Figure 14

Glen Loch bridge, Pennsylvania

Year	1912
Type	Rail
Country	United States
Cause	Limited knowledge
Details	Fatigue
Fatalities	4
Injuries	0
Collapse	complete
Phase	in service
Reference	SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Coos-Bay-bridge Oregon

Year 1924

Type Rail

Country United States

Cause Impact

Details Ship impact underside of deck

Fatalities 0

Injuries 0

Collapse partial

Phase in service

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Figure 15

Ohio Falls truss bridge

Year	1927
Type	Rail
Country	United States
Cause	Design error
Details	Insufficient bracing of intermediate scaffolding (cantilevered construction)
Fatalities	1
Injuries	0
Collapse	partial
Phase	construction
Continent	North America
Reference	SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000

Poughkeepsie suspension bridge

Year 1927

Type Road

Country United States

Cause Human error

Details Quality of ground much worse than expected

Fatalities 0

Injuries 0

Collapse partial

Phase construction

Continent North America

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Figure 16

4-span beam and slab bridge (Anacostia River)

Year 1933

Type Rail

Country United States

Cause Natural hazard (Flooding)

Details Scour, lacking inspection

Fatalities 0

Injuries 0

Collapse partial

Phase in service

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Steel truss bridge near Manassas, Virginia

Year 1937

Type Road

Country United States

Cause Impact

Details Truck impact on compression strut of the truss

Fatalities 0

Injuries 0

Collapse complete

Phase in service

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Truss bridge near Pagosa Springs in Colorado

Year 1937

Type Road

Country United States

Cause Overloading

Details High concentration of vehicles

Fatalities 0

Injuries 0

Collapse complete

Phase in service

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Whiteson Bridge near Minnville, Oregon (North Yamhill River)

Year 1937

Type Road

Country United States

Cause Impact

Details Height of truck bigger than maximum headroom of portal frame

Fatalities 0

Injuries 0

Collapse complete

Phase in service

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Bridge near the Niagara Falls (Niagara River)

Year 1938

Type Road

Country United States

Cause Natural hazard (Debris in water)

Details Ice-induced pressure on arch abutments leads to collapse

Fatalities 0

Injuries 0

Collapse complete

Phase in service

References SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.
BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002.
http://www.iti.nwu.edu/links/bridges/disasters.html

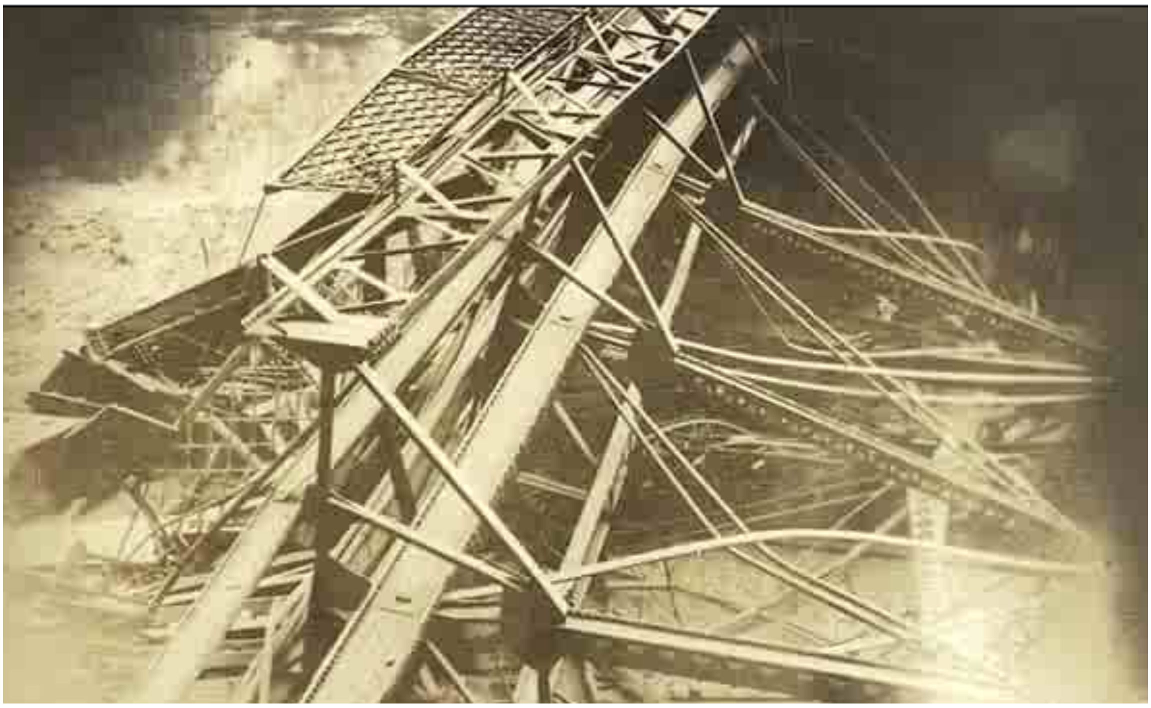


Figure 17

Figure 18

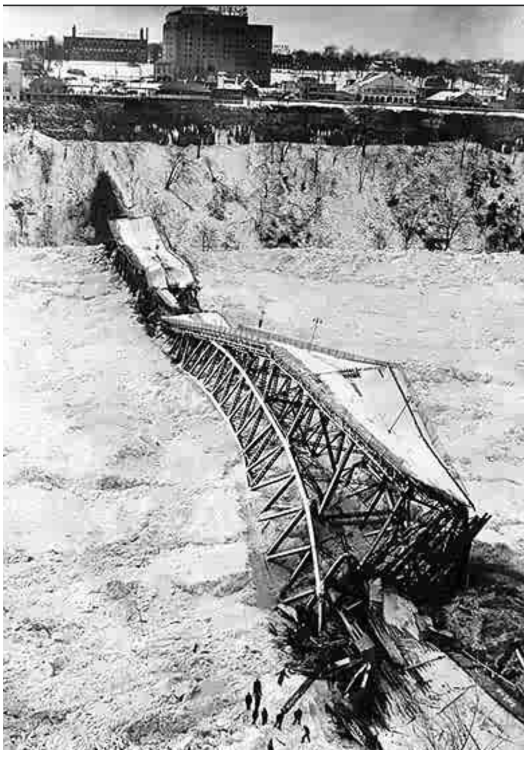


Figure 19

Figure 20

Battered and twisted, its steel girders tangled grotesquely, the shimmering Falls View "honeymoon" bridge linking the United States and Canada across the Niagara river, today emerged the victor in a dramatic battle with a huge ice jam which, as the directed by an unseen hand, sought inexorably to destroy the graceful span. A sharp drop in temperature, tumbling the mercury to eight above zero, coupled with the ceaseless toil of hastily recruited workers, relieved pressure of the 100-foot ice pack at the foundations of the bridge this morning. Triumphant but weary, owners of the structure declared that danger of collapse had passed, at least temporarily. Toiling under floodlights thruout[sic] the night, laborers with picks and shovels dug a 60-foot ditch around abutments on the American side. When clear water was reached wooden bulwarks were placed against the sorely torn girders. Receding temperatures checked the flow of ice over the fall and indications were that the lower river current was gradually carrying away much ice from under the jam. The tremendous force of the floes have pushed piles of ice 100 feet high up under the bridge. Iron girders were bent and twisted like wire. The deck of the bridge reared ten feet in the air yesterday afternoon. The main lower arch girder of the bridge seems bent and twisted beyond hope of repair. The deck of the bridge sagged several feet from the American side. The base of support on this side seems moved about 7 feet from its original resting place. The Falls View bridge is one of three structures spanning the gorge near the falls.

Tacoma Narrows suspension bridge

Year 1940

Type Road

Country United States

Cause Limited knowledge

Details Insufficient bending- and torsion stiffness, aerodynamic instability

Fatalities 0

Injuries 0

Collapse complete

Phase in service

Continent North America

References SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.
LEVY, M., SALVADORI, M., Why Buildings Fall Down, W.W. Norton & Company, New York, 2002.
BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002.
http://www.basedn.freeserve.co.uk/bridge.htm
http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php
http://www.iti.nwu.edu/links/bridges/disasters.html

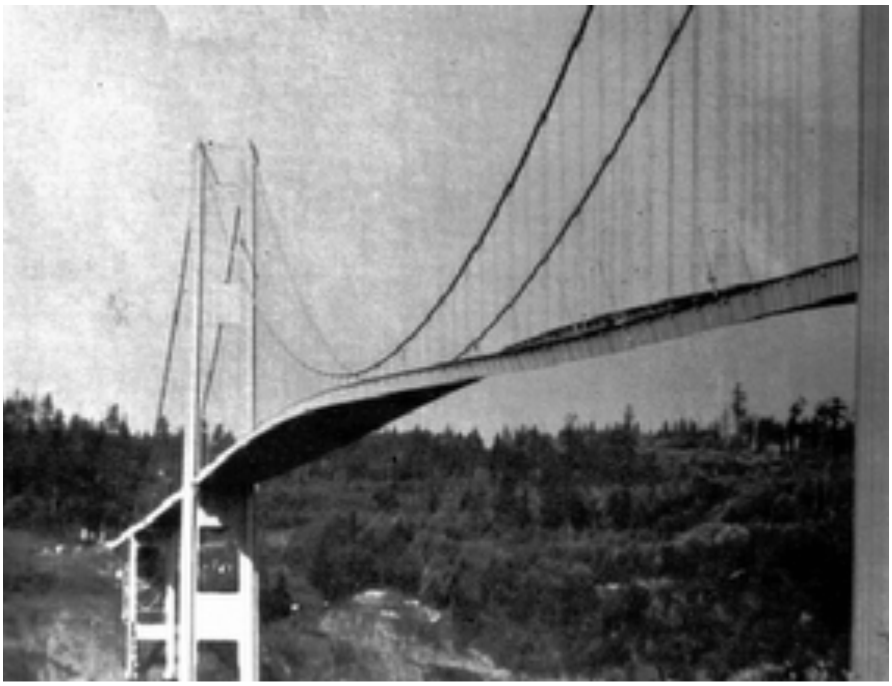


Figure 21

Length of center span	2800 ft
Width	39 ft
Depth of stiffening girders	8 ft
Start of construction	Nov, 23, 1938
Opened for traffic	July, 1, 1940
Collapse of bridge	Nov, 7, 1940

Figure 22

The first Tacoma Narrows Bridge opened to traffic on July 1, 1940. Its main span collapsed into the Tacoma Narrows four months later on November 7, 1940, at 11:00 AM (Pacific time) due to a physical phenomenon known asaeroelastic flutter caused by a 67 kilometres per hour (42 mph) wind. The bridge collapse had lasting effects on science and engineering. In many undergraduate physics texts the event is presented as an example of elementary forced resonance with the wind providing an external periodic frequency that matched the natural structural frequency (even though the real cause of the bridge's failure was aeroelastic flutter[1]). Its failure also boosted research in the field of bridge aerodynamics/aeroelastics which have themselves influenced the designs of all the world's great long-span bridges built since 1940.

WHY CODES DON'T COVER EXCEPTIONAL LARGE SCALE BRIDGES?

Plate girder Gerber hinge bridge near New York

Year 1939

Type Road

Country United States

Cause Impact

Details Ship impact of ship with loose anchor

Fatalities 0

Injuries 0

Collapse partial

Phase construction

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

2-span truss bridge over Mississippi in Chester, Illinois

Year 1944

Type Road

Country United States

Cause Design error

Details Uplifting wind load not considered

Fatalities 0

Injuries 0

Collapse complete

Phase in service

References SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.
BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002.
SMITH, D.W., Bridge failures, Proceedings of the Institution of Civil Engineers, Part 1, Vol. 60, 1976, pp. 367-382.

Swing bridge in Boston-Charlestown, Massachusetts

Year 1945

Type Road

Country United States

Cause Impact

Details Ship impact in half-open swing bridge

Fatalities 0

Injuries 0

Collapse partial

Phase in service

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

John Grace-Memorial Bridge (Cooper River), South Carolina

Year 1946

Type Road

Country United States

Cause Impact

Details Ship forced by wind into bridge deck

Fatalities 0

Injuries 0

Collapse partial

Phase in service

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Bridge near Fresno, California (King's Slough River)

Year 1947

Type Road

Country United States

Cause Overloading

Details Agriculture vehicle train

Fatalities 0

Injuries 0

Collapse complete

Phase in service

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000

Rockport-Bridge, Maine (Goose River)

Year 1947

Type Road

Country United States

Cause Impact

Details Truck impact on truss

Fatalities 0

Injuries 0

Collapse complete

Phase in service

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Hinton truss bridge, West Virginia

Year 1949

Type Road

Country United States

Cause Design error

Details Insufficient capacity of cantilever arm during construct

3ion phase

Fatalities 5

Injuries 4

Collapse partial

Phase construction

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Elbow Grade Bridge, Willamette National Forest, timber truss

Year 1950

Type Road

Country United States

Cause Design error

Details Parts of truss underdesigned

Fatalities 0

Injuries 0

Collapse complete

Phase in service

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Sullivan Square motorway bridge, Boston

Year 1952

Type Road

Country United States

Cause Design error

Details Instability of scaffolding

Fatalities 0

Injuries 0

Collapse complete

Phase construction

Reference BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002.

Eric bridge, Cleveland, Ohio

Year 1956

Type Rail

Country United States

Cause Natural hazard (Wind)

Details Winds

Fatalities 0

Injuries 0

Collapse complete

Phase in service

Reference HARIK, I.E., SHAABAN, A.M., GESUND, H., VALLIS, G.Y.S., WANG, S.T., United States Bridge Failures, 1951-1988, Journal of Performance of Constructed Facilities, Vol. 4, No. 4, 1990.

Interstate 29 West Bridge, Sioux City, Iowa

Year 1962

Type Road

Country United States

Cause Natural hazard (Flooding)

Details Scour

Fatalities 0

Injuries 0

Collapse complete

Phase in service

Reference http://www.e-commatrix.com/PBB/dnb-copy.html

Lake Pontchartrain bridge (Lake Pont)

Year 1964

Type Road

Country United States

Cause Impact

Details Ship impact, error of ship captain

Fatalities 6

Injuries 0

Collapse partial

Phase in service

References SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.
http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php
SMITH, D.W., Bridge failures, Proceedings of the Institution of Civil Engineers, Part 1, Vol. 60, 1976, pp. 367-382.

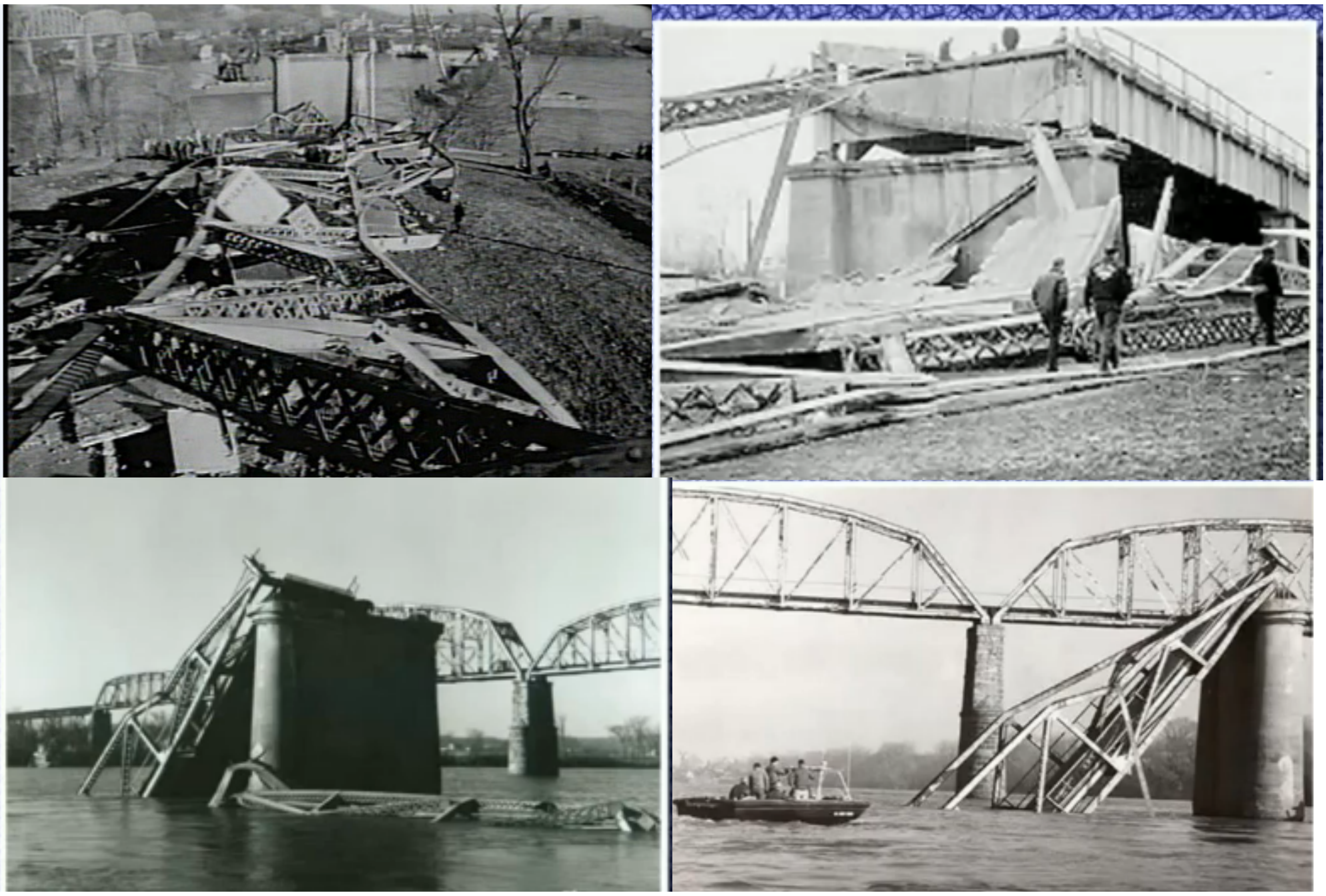
Bridge near Charleston, South Carolina (Cooper River)

Year	1965
Type	Rail
Country	United States
Cause	Natural hazard (Flooding)
Details	Scour, pier failure
Fatalities	0
Injuries	0
Collapse	no
Phase	in service
Reference	SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Silver bridge, chain suspension bridge (Ohio River)

Year	1967
Type	Road
Country	United States
Cause	Limited knowledge
Details	Fatigue
Fatalities	46
Injuries	9
Collapse	complete
Phase	in service
References	SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000. BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002. http://www.englib.cornell.edu/ice/lists/historytechnology/successfailures.html http://www.iti.nwu.edu/links/bridges/disasters.html MENZIES, J.B., Bridge Safety Targets, Report for the Highways Agency, Ref : HA5021C, 1996.

Figure 23



Figures 24-25-26-27

A lot of people have already heard about the Silver Bridge and how it collapsed in mysterious. The Silver Bridge made the intersection of Main and Sixth streets one of the busiest in Point Pleasant.

The bridge was dubbed the ‘Silver Bridge‘ because it was the country’s first aluminum painted bridge. It was constructed in 1928 and consisted of a 700 foot center span and 380 foot side spans. It was of suspension design with “eye bars” chained and linked together with massive pins, instead of the conventional wire cables. Then Silver bridge was the first eye-bar suspension bridge of its type to be constructed in the United States.

On December 15,1967 at approximately 5 p.m., the U.S. Highway 35 bridge connecting Point Pleasant, West Virginia and Kanauga, Ohio suddenly collapsed into the Ohio River. The structure only took about 1 minute to completely fall into the river below. Dozens of cars and trucks followed the structure into the river, claiming 46 lives and 9 injuring.

Bridge in Illinois (Kaslaski River)

Year 1970

Type Rail

Country United States

Cause Design error

Details Not anchored against uplift

Fatalities 0

Injuries 0

Collapse complete

Phase in service

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Buckman Bridge near Jacksonville, Florida

Year 1970

Country United States

Cause Limited knowledge

Details Voided pier fills with sea water during construction, anaerobic bacteria produce methan gas --> expansion of pier --> partial collapse of bridge

Fatalities 0

Injuries 0

Collapse partial

Phase in service

Continent North America

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Chesapeake Bay Bridge, Annapolis

Year 1970

Country United States

Cause Impact

Details Military ship gets out of control and hits the bridge during one hour and stormy weather, 5 spans collapse, 11 other spans damaged

Fatalities 0

Injuries 0

Phase in service

Reference http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php

Motorway bridge, Junction Antelope Valley

Year 1971

Type Road

Country United States

Cause Natural hazard (Earthquake)

Details Earthquake

Fatalities 0

Injuries 0

Collapse partial

Phase in service

Reference BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002.

Motorway bridge near Pasadena, California (Arroyo Seco River)

Year 1972

Type Road

Country United States

Cause Design error

Details Scaffolding collapses under weight of fresh concrete

Fatalities 6

Injuries 0

Collapse partial

Phase construction

References SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.
SMITH, D.W., Bridge failures, Proceedings of the Institution of Civil Engineers, Part 1, Vol. 60, 1976, pp. 367-382.

Chesapeake Bay Bridge, Annapolis

Year 1970

Country United States

Cause Impact

Details Military ship gets out of control and hits the bridge during one hour and stormy weather, 5 spans collapse, 11 other spans damaged

Fatalities 0

Injuries 0

Collapse partial

Phase in service

Reference http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php

Sidney-Lanier Bridge Brunswick, Georgia

Year 1972

Type Road

Cause Impact

Details Ship impact, misunderstanding captain - staff

Fatalities 10

Injuries 0

Collapse partial

Phase in service

References SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.
http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php
SMITH, D.W., Bridge failures, Proceedings of the Institution of Civil Engineers, Part 1, Vol. 60, 1976, pp. 367-382.

The ship noticed that it was heading into the bridge so the ship dropped its anchor, but the anchor failed to holdWhen the ship hit the bridge, it knocked several spans apart. Ten to fifteen automobiles fell into the water when the spans fell apart

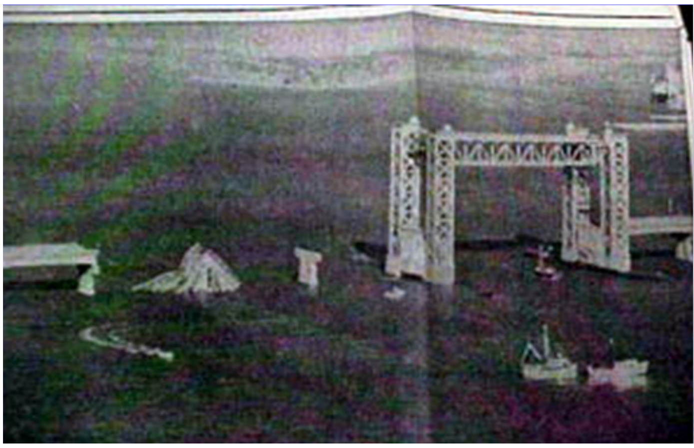


Figure 28

Figure 29

Lake Pontchartrain bridge (Lake Pont)

Year 1974

Type Road

Cause Impact

Details Ship impact, captain slept

Fatalities 3

Injuries 0

Collapse partial

Phase in service

References SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.
http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php
SMITH, D.W., Bridge failures, Proceedings of the Institution of Civil Engineers, Part 1, Vol. 60, 1976, pp. 367-382.

Figure 30

3-span bridge in Lafayette Street, St-Paul, Minnesota

Year 1975

Type Road

Country United States

Cause Limited knowledge

Details Brittle failure of new steel

Fatalities 0

Injuries 0

Collapse no

Phase in service

References SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.
BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002.

21-span, Pass Manchac Bridge, Louisiana

Year 1976

Type Road

Country United States

Cause Impact

Details Ship impact, error of ship captain

Fatalities 2

Injuries 2

Collapse partial

Phase in service

References SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.
http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php

Figure 31

Fulton Yates Bridge near Henderson, Kentucky

Year	1976
Type	Road
Country	United States
Cause	Overloading
Details	Overloading during refurbishment
Fatalities	0
Injuries	0
Phase	in service
References	SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000. http://www.e-commatrix.com/PBB/dnb-copy.html

Benjamin Harrison Memorial Bridge near Hopewell, Virginia

Year	1977
Type	Road
Country	United States
Cause	Impact
Details	Ship impact, failure in electronic of ship guidance
Fatalities	0
Injuries	0
Collapse	partial
Phase	in service
References	SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000. http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php

Bridge over Passiac River, Union Avenue

Year	1977
Country	United States
Cause	Impact
Details	Ship impact, 2 spans collapse
Fatalities	0
Injuries	0
Phase	in service
Reference	http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php

Interstate 17 Bridge, Black Canyon, Arizona

Year	1978
Type	Road
Country	United States
Cause	Natural hazard (Flooding)
Details	Flood
Fatalities	0
Injuries	0
Collapse	complete
Phase	in service
Reference	http://www.e-commatrix.com/PBB/dnb-copy.html

Southern Pacific Railroad Bridge (Berwick Bay), Louisiana

Year	1978
Type	Rail
Country	United States
Cause	Impact
Details	Ship impact, steel truss of 70 m falls into water and sinks
Fatalities	0
Injuries	0
Phase	in service
Reference	http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php

K&I Railroad Bridge, Louisville, Jefferson County, Kentucky

Year	1979
Type	Rail
Country	United States
Cause	Overloading
Details	Vehicle exceeding weight limit
Fatalities	0
Injuries	0
Collapse	complete
Phase	in service
Reference	http://www.e-commatrix.com/PBB/dnb-copy.html

Alabama Rail Bridge, Alabama

Year	1979
Type	Rail
Country	United States
Cause	Impact
Details	Train impact
Fatalities	0
Injuries	0
Collapse	complete
Phase	in service
Reference	http://www.e-commatrix.com/PBB/dnb-copy.html

Southern Rail Bridge, Indiana

Year	1979
Type	Rail
Country	United States
Cause	Overloading
Details	Vehicle exceeding weight limit
Fatalities	0
Injuries	0
Collapse	complete
Phase	in service
Reference	http://www.e-commatrix.com/PBB/dnb-copy.html

Bridge over the Hood canal, Washington

Year	1979
Country	United States
Cause	Natural hazard (Wind)
Details	Wind and storm
Fatalities	0
Injuries	0
Collapse	complete
Phase	in service
Reference	BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002.

Interstate 10 Bridge, Phoenix, Arizona

Year	1979
Type	Road
Country	United States
Cause	Natural hazard (Flooding)
Details	Flood
Fatalities	0
Injuries	0
Collapse	partial
Phase	in service
Reference	http://www.e-commatrix.com/PBB/dnb-copy.html

Concrete 5-span box girder bridge near Rockford

Year	1979
Type	Road
Country	United States
Cause	Design error
Details	Big cracks, failure of Epoxy-filled joint (not enough hardened to take shear force)
Fatalities	0
Injuries	0
Collapse	no
Phase	construction
Reference	SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Sunshine Skyway Bridge near St.Petersburg, Florida

Year 1980

Type Road

Country United States

Cause Impact

Details Ship impact, not enough care of captain in bad weather

Fatalities 35

Injuries 0

Collapse partial

Phase in service

References SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.
http://www.basedn.freeserve.co.uk/bridge.htm
http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php
http://www.geocities.com/pagesbydave/SunSkyDemoHis.html

Figure 32

Figure 34

Opening of the newer span was delayed until 1971 for reinforcing of the south main pier, which had cracked due to insufficient supporting pile depth. The second span was used for all southbound traffic, while the original span was converted to carry northbound traffic.

The southbound span (opened in 1971) of the original bridge was destroyed at 7:33 a.m. on May 9, 1980, when the freighter MV Summit Venture collided with a pier (support column) during a storm, sending over 1200 feet (366m) of the bridge plummeting into Tampa Bay. The collision caused six automobiles and a Greyhound bus to fall 150 feet (46 m), killing 35 people.

Ironically, the south main pier (the one that required reinforcement before completion) withstood the ship strike without significant damage. It was the second pier to the south of it that was destroyed, a secondary pier that was not designed to withstand a large ship strike.

Figure 35

Truss bridge in Trenton, Wisconsin (Milwaukee River)

Year	1980
Type	Road
Country	United States
Cause	Impact
Details	Truck impact on main truss
Fatalities	0
Injuries	1
Collapse	complete
Phase	in service
Reference	SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Multiple span box girder bridge in East Chicago, Indianapolis

Year 1982

Type Road

Country United States

Cause Design error

Details Scaffolding collapses under weight of fresh concrete

Fatalities 13

Injuries 18

Collapse partial

Phase construction

References SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.
BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002.
http://www.in.gov/dot/programs/inmemoriam/35.html
http://www.basedn.freeserve.co.uk/bridge.htm
http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php

Figure 36

Before the construction of the expressway, portions of Truck Route 912 were on Kennedy Avenue. 5.7 miles (9.2 km) of new expressway from the Toll Road to Chicago Avenue was constructed. On April 15, 1982, fourteen workers were killed and eighteen injured when falsework beneath a ramp failed during a concrete pour.

OSHA discovered several errors that caused the collapse of the bridge section. The most likely cause of the collapse was "the cracking of a concrete pad supporting a leg of the shoring towers." The failure of the concrete pad, built too thin, led to another finding; 1 inch bolts that were supposed to connect key stringers to cross-beams instead were replaced with frictional clips, but investigators did not find any documentation that supported this substitution. Investigators could not locate any engineering calculations supporting the pads as designed; worse, the pads were built substandard to the undocumented design.

Prestressed concrete precast box girder bridge, Saginaw

Year 1982

Type Road

Country United States

Cause Design error

Details Too weak temporary support elements

Fatalities 0

Injuries 0

Collapse partial

Phase construction

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Syracuse bridge, New York

Year 1982

Type Road

Country United States

Cause Design error

Details Torsional buckling due to lacking lateral support

Fatalities 1

Injuries 5

Collapse partial

Phase construction

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

Connecticut Turnpike Bridge near Greenwich (Mianus River)

Year 1983

Type Road

Country United States

Cause Deterioration

Details Corrosion of joint hangers (Gerber-joint), constraint stresses due to big skewness

Fatalities 3

Injuries 3

Collapse partial

Phase in service

References SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.
LEVY, M., SALVADORI, M., Why Buildings Fall Down, W.W. Norton & Company, New York, 2002.
BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002.
http://www.englib.cornell.edu/ice/lists/historytechnology/successfailures.html
http://www.iti.nwu.edu/links/bridges/disasters.html
MENZIES, J.B., Bridge Safety Targets, Report for the Highways Agency, Ref : HA5021C, 1996.

Figure 37

The Mianus River Bridge carries Interstate 95 over the Mianus River in the Cos Cob section of Greenwich, Connecticut. The bridge had a 100-foot section of its deck of its northbound span collapse on June 28, 1983. Three people were killed when their vehicles fell with the bridge into the Mianus River 70 feet below, and three were seriously injured. Casualties from the collapse were few because the disaster occurred at 1:30 a.m., when traffic was low on the often crowded highway.

The collapse was caused by the failure of two pin and hanger assemblies that held the deck in place on the outer side of the bridge. The hanger on the inside part of the expansion joint at the southeast corner was forced from the pin that was holding it, and the load was shifted to the only other pin in the joint. The problem was caused by rust formation within the bearing on the pin, exerting a tremendous force on the hanger. The extra load on the remaining pin started a fatigue crack at a sharp corner on the pin. When it failed catastrophically, the deck was supported at just three corners. When two heavy trucks and a car entered the section, the remaining expansion joint failed, and the deck crashed into the river below.

The ensuing investigation cited corrosion from water buildup due to inadequate drainage as a cause. During road mending some 10 years before, the highway drains had been deliberately blocked and the crew failed to unblock them when the road work was completed. Rainwater leaked down through the pin bearings, causing them to rust. The outer bearings were fracture-critical and non-redundant, a design flaw of this particular type of structure. The bearings were difficult to inspect close-up, although traces of rust could be seen near the affected bearings.

The incident was also blamed on inadequate inspection resources in the state of Connecticut. At the time of the disaster, the state had just 12 engineers, working in pairs, assigned to inspect 3,425 bridges. The collapse came despite the nationwide inspection procedures brought about by the collapse of the Silver Bridge in West Virginia in December 1967.

Walnut street viaduct over Interstate 20 in Denver, Colorado

Year	1985
Type	Road
Country	United States
Cause	Design error
Details	Failure of pier head sending eight 55-ton bridge girders onto road underneath
Fatalities	1
Injuries	4
Collapse	partial
Phase	construction
References	SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000. http://www.denverpost.com/Stories/0,1413,36~53~2151748,00.html

Walnut Street Viaduct over I-25 in Denver, under construction, collapsed during the early morning hours of Oct. 4, 1985, killing one construction worker and injuring four others. The new viaduct was being built to replace the old Larimer Street Viaduct.

The Mousetrap, so dubbed by a radio reporter in the 1960s, is the intersection of I-25 and I-70 northwest of downtown Denver . It was so called because it supposedly looked like a mousetrap from the air, and because it was infamous for trapping vehicles within its network of sharply curving ramps.

In the early-morning hours of August 1, 1984, a truck carrying a load of torpedoes on I-25 exited onto one of the offramps and, apparently traveling and too high a speed, dumped the load under the various Mousetrap structures. The freeway was closed for miles in all directions for several hours. This prove the indedequacy to handle the estimated 300,000 vehicles daily.

In 1964, the portion of newly-designated (1957) I-70 from Colorado Boulevard west to I-25 was opened on an elevated structure above 46 th Ave. A modified interchange at I-25 was built (this interchange had previously served 46 th Ave.). One year later, in 1965, the portion of I-70 going west from I-25 was opened. This portion was aligned with 48 th Ave.

Following the “Torpedoes in the Mousetrap” incident in 1984, plans were made to reconstruct the Mousetrap and the portion of I-25 that was first to open in Denver in 1951.

Bridge in El Paso, Texas

Year 1987

Country United States

Cause Design error

Details Inadequate scaffolding

Fatalities 1

Injuries 7

Phase construction

Reference BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002.

Schoharie Creek Bridge (New York)

Year 1987

Type Road

Country United States

Cause Natural hazard (Flooding)

Details Flooding and storm lead to collapse of two spans after scouring of a pier

Fatalities 10

Injuries 0

Collapse partial

Phase in service

References SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.
LEVY, M., SALVADORI, M., Why Buildings Fall Down, W.W. Norton & Company, New York, 2002.
BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002.
http://www.iti.nwu.edu/links/bridges/disasters.html

The Schoharie Creek Bridge was a New York State Thruway bridge over the Schoharie Creek near Fort Hunter, in New York State. On April 5, 1987 it collapsed due to erosion at the foundations after a record rainfall. The collapse killed ten people.

The final design for the bridge was approved in January 1952 by the New York State Department of Transportation, (previously named The New York State Department of Public Works). The design described 509 ft crossing consisting of five simply supported spans with nominal lengths of 100 ft, 110 ft, 120 ft, 110 ft, and 100 ft. The bridge was supported with pier frames along with abutments at each end. The pier frames were constructed of two slightly tapered columns with tie beams. The columns were fixed in place within a lightly reinforced plinth positioned on a shallow, reinforced spread footing. The spread footing was to be protected with a dry layer of riprap.

The superstructure consisted of two longitudinal main girders with transverse floor beams. The skeleton of the bridge deck, 7.9 in thick, was made up of steel stringers.

Motorway bridge near Seattle

Year 1988

Type Road

Country United States

Cause Design error

Details Girders not yet tied together by diaphragms, Domino effect

Fatalities 0

Injuries 0

Phase construction

Reference BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002.

Box girder bridge in Los Angeles

Year 1989

Type Road

Country United States

Cause Design error

Details Collapse when scaffolding was removed

Fatalities 0

Injuries 5

Phase construction

Reference BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002.

Bridge in Baltimore

Year	1989
Type	Road
Country	United States
Cause	Design error
Details	Prestressing not in place, asymmetric loading
Fatalities	1
Injuries	14
Phase	construction
Reference	BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002.

Figure 38

Cypress Freeway, Oakland, California

Year 1989

Type Road

Country United States

Cause Natural hazard (Earthquake)

Details Loma Prieta earthquake

Fatalities 42

Injuries 0

Collapse complete

Phase in service

Reference http://www.sfmuseum.net/cypress/response.html

Figure 39

Figure 40

The Cypress Street Viaduct, often referred to as the Cypress Structure, was a 1.6 mile long, raised two-tier, multi-lane (four lanes per deck) freeway constructed of reinforced concrete that was originally part of the Nimitz Freeway (State Highway 17, and later, Interstate 880) in Oakland, California.

It replaced an earlier single-deck viaduct constructed in the 1930s as one of the approaches to the San Francisco–Oakland Bay Bridge.

It officially opened to traffic on June 11, 1957 and was in use until the Loma Prieta Earthquake occurred on October 17, 1989, when much of the upper tier collapsed onto the lower tier resulting in 42 fatalities.

The double-decked viaduct was initially designed in 1949 by the City of Oakland as a way to ease traffic on local streets leading to the Bay Bridge, such as Cypress Street (which was California State Route 17 at the time).

On October 17, 1989, the portion of the structure from 16th Street north all the way to the MacArthur Maze collapsed during the Loma Prieta Earthquake, due to ground saturation and structural flaws. When it was in use, the upper tier was used by southbound traffic, and the lower tier was used by northbound traffic. Some sections of the Cypress Street Viaduct were largely supported by two columns on either side, but some sections were only supported beneath by a single supporting column. The design was unable to survive the earthquake because the upper portions of the exterior columns were not tied by reinforcing to the lower columns, and the columns were not sufficiently ringed to prevent bursting. At the time of its design, such structures were not analyzed as a whole, and it appears that large structure motion contributed to the collapse. It was built on filled land, which is highly susceptible to soil liquefaction during an earthquake and exhibits larger ground motion.

After the earth stopped moving, local residents and workers began crawling into and climbing upon the shattered structure with the goal of rescuing those left alive. Many were saved; some only by amputation of trapped limbs.[2] The collapse of the upper tier onto the lower tier resulted in 42 fatalities—two-thirds of the total quake death toll of 63[3].

The viaduct was torn down, Cypress Street was renamed (now known as Mandela Parkway, in honor of Nelson Mandela) with a landscaped median planted where the viaduct once stood. Before reconstruction occurred, the viaduct ended at the Eighth Street exit on the southern end, with the two roadways going over Seventh Street, while the southbound exit off the MacArthur Maze onto Cypress Street at 32nd Street remained open to local traffic on the northern end.

Figure 41

Section of East span of San Francisco Oakland Bay Bridge, California

Year	1989
Type	Road
Country	United States
Cause	Natural hazard (Earthquake)
Details	Loma Prieta earthquake
Fatalities	1
Injuries	0
Collapse	partial
Phase	in service
References	BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002. http://www.iti.nwu.edu/links/bridges/disasters.html

Figure 42

Truss bridge in Shepherdsville, Kentucky

Year 1989

Type Rail

Country United States

Cause Impact

Details Litter collector is higher than bridge clearance

Fatalities 0

Injuries 0

Collapse partial

Phase in service

References SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.
MENZIES, J.B., Bridge Safety Targets, Report for the Highways Agency, Ref : HA5021C, 1996.

Herbert C. Bonner Bridge, North Carolina

Year 1990

Country United States

Cause Impact

Details Ship impact, 4 piers damaged, 5 spans collapse

Fatalities 0

Injuries 0

Phase in service

Reference http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php

Motorwaybridge, jonction Antelope Valley

Year 1992

Type Road

Country United States

Cause Natural hazard (Earthquake)

Details One span collapses during earthquake

Fatalities 0

Injuries 0

Collapse partial

Phase in service

Reference BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002.

Truss bridge in Concord, New Hampshire

Year 1993

Type Road

Country United States

Cause Human error

Details Stiffener mounted mounted at wrong place

Fatalities 2

Injuries 7

Collapse complete

Phase construction

Reference SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000

Truss bridge near Mobile CSXT Big Bayou Canot, Alabama

Year	1993
Type	Rail
Country	United States
Cause	Impact
Details	Ship impact
Fatalities	47
Injuries	103
Collapse	partial
Phase	in service
References	SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000. http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php

The 1993 Big Bayou Canot train wreck was the derailing of an Amtrak train on the CSXT Big Bayou Canot bridge in northeast Mobile, Alabama, USA, killing 47 and injuring 103, on September 22,1993. It is the deadliest train wreck in the history of the United States passenger railroad company Amtrak.

The collision forced the bridge approximately three feet out of alignment and severely kinked the track. The bridge span had actually been designed to rotate so it could be converted to a swing bridge by adding suitable equipment. No such conversion had ever been performed, but the span had not been adequately secured against unintended movement.

Interstate 5 Bridge in Los Angeles, California

Year	1994
Type	Road
Country	United States
Cause	Natural hazard (Earthquake)
Details	Earthquake measuring 6.6 on the Richter scale
Fatalities	57
Injuries	0
Collapse	partial
Phase	in service
Reference	24 die as quake strikes LA, The Guardian, 18 January 1994, p.11+24.

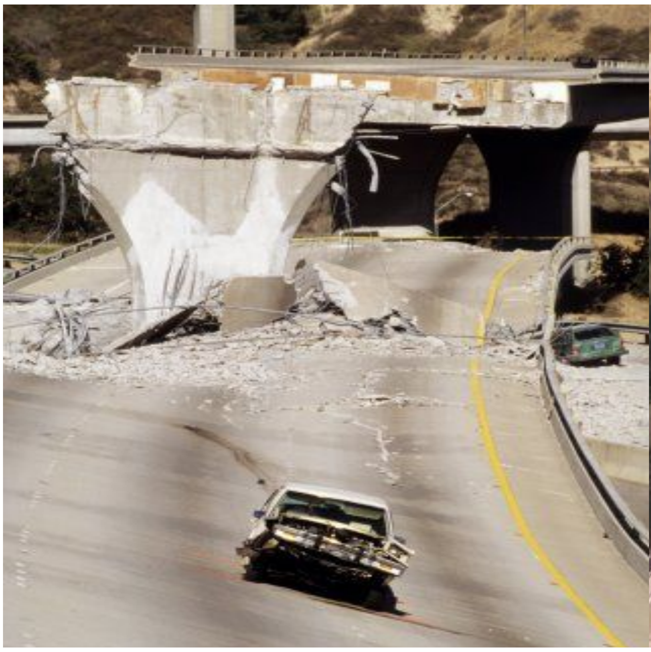


Figure 43

Figure 44

January 17, 1994, Northridge earthquake, which caused 57 deaths and widespread damages.

3-span 3-girder composite bridge near Clifton (Tennessee River)

Year	1995
Type	Road
Country	United States
Cause	Human error
Details	Executed construction sequence different from planned one
Fatalities	1
Injuries	0
Collapse	complete
Phase	construction
References	SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000. http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php

Twin bridges, Interstate 5 (Arroyo Pasajero River), Coalinga, California

Year	1995
Type	Road
Country	United States
Cause	Natural hazard (Flooding)
Details	Scour of bridge foundations
Fatalities	7
Injuries	0
Collapse	complete
Phase	in service
References	BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002. http://www.tfhrc.gov/pubrds/fall95/p95a2.htm http://www.iti.nwu.edu/links/bridges/disasters.htm

Figure 45

Bridge near Covington, Tennessee (Hatchie River)

Year	1999
Type	Road
Country	United States
Cause	Natural hazard/Change in structural enviroment (Flooding)
Details	Scouring and undermining of the foundations
Fatalities	8
Injuries	0
Collapse	partial
Phase	in service
References	LEVY, M., SALVADORI, M., Why Buildings Fall Down, W.W. Norton & Company, New York, 2002. MENZIES, J.B., Bridge Safety Targets, Report for the Highways Agency, Ref : HA5021C, 1996.

85-foot section of the bridge fell into the rain-swollen Hatchie River due to the rushing water that had weakened bridge supports. Four passenger cars and a tractor-trailer rig plunged into the river, killing all occupants. Eight people were killed.

A federal investigation found that the river channel had moved 83 feet since the bridge was built in 1936, and that the bridge likely failed as a result of the deterioration of timber piles that were originally buried and not designed to be in water.

Figure 46

Bridge over motorway in Concord (North Carolina)

Year 2000

Type Foot/pedestrian

Country United States

Cause Overloading

Details Bridge snapped in half as tens of thousands of people left a motor speedway event on Saturday evening and were crossing over it

Fatalities 0

Injuries 107

Collapse complete

Phase in service

References http://www.basedn.freeserve.co.uk/bridge.htm
http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php
CEDERQUIST S. C. , Motor speedway bridge collapse caused by corrosion, Materials Performance, Vol. 39, No. 7, 2000, pp. 18-19

Queen Isabella Causeway, Texas

Year 2001

Type Road

Country United States

Cause Impact

Details Four barges and a tugboat struck the bridge

Fatalities 8

Injuries 0

Collapse partial

Phase in service

Reference http://news.bbc.co.uk/1/hi/world/americas/2009472.stm

Four loaded barges crashed into one of the support columns traveling at 2/10ths of 1 mile per hour. Three 80-foot sections of the bridge fell into the water, leaving a large gap in the roadway. The collapsed sections were just next to the highest point of the causeway, making it difficult for approaching drivers to notice. Eight people were killed as their cars fell 85 feet into the water. Five vehicles were recovered from the water along with three survivors.

Figure 47

Historic Tewksbury Township pony truss bridge, Hunterdon County, New Jersey

Year	2001
Country	United States
Cause	Impact
Details	30-ton tractor-trailer ignored signs warning of the bridge's 8-ton weight limit. Truck struck the bridge abutment and caused it to collapse.
Fatalities	0
Injuries	0
Collapse	complete
Phase	in service
Reference	http://www.nj.com/news/expresstimes/nj/index.ssf?/base/news-5/1091264704141461.xml

Interstate 40 Bridge, Oklahoma (Webber Falls)

Year 2002

Type Road

Country United States

Cause Impact

Details Ship collides with one of piers, bridge collapses on length of 150 m

Fatalities 14

Injuries 4

Collapse partial

Phase in service

Reference http://www.brueckenweb.de/Themen/katastrophen/katastrophen.php

A barge collided with the Interstate 40 Bridge near Webbers Falls, Oklahoma, collapsing multiple spans and killing 14 people.

Figure 48

Figure 49

Marcy bridge (Utica-Rome Expressway project)

Year 2002

Type Foot

Country United States

Cause Design error

Details Global torsional buckling, bridge not braced properly as workers built it. The braces could not hold the long, narrow bridge as workers poured the concrete deck onto it.

Fatalities 1

Injuries 9

Collapse complete

Phase construction

Reference http://www.uticaod.com/archive/2004/07/01/news/36576.html

Figure 50

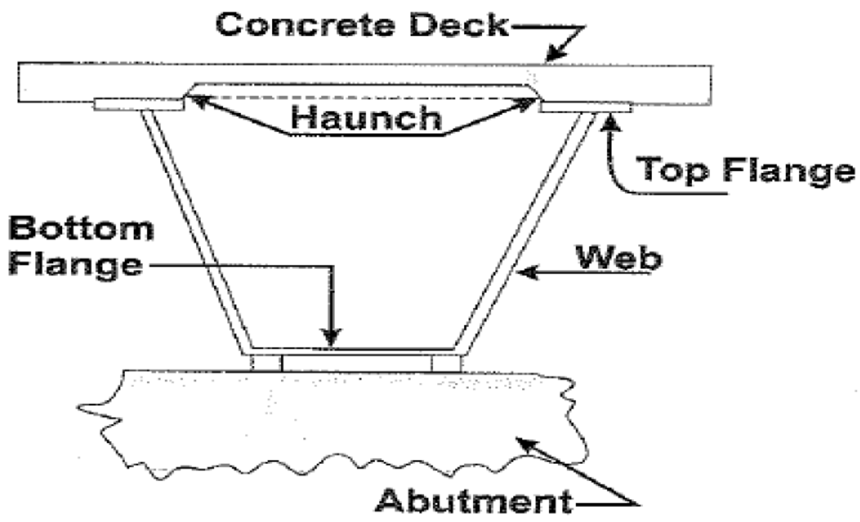


Figure 51

Collapse of the Marcy Pedestrian Bridge during the construction phase due to Lateral-Torsional-Buckling (LTB) during the deck concreting phase, which killed a worker.

The responsibility was first to the designer that designed the girder and its bracing without taking into account the global torsional buckilng (un conservative choice).

Since the bridge is classified as a single box girder, the designed girder was excluded from the governing code subsection 10.51, which provides specifications for two or more single cell composite box girders. Only in this subsection the buckling phenomena were mentioned, although the global torsional buckling was not considered. Moreover the codes address the topic of bracing only with regards to bridges which have been already erected and bracers are specified to support flanges that will never be made composite. Consequently, since the considered bridge collapsed during the execution and is a composite structure these codes can't be considered valid for it.

Though diaphragms can't be considered as bracing systems if not anchored, this distinction is not clearly expressed in the NYCDOT code, where diaphragms are called bracing.

Figure 52

Figure 53

1900 built Kinzua Viaduct (north-central Pennsylvania), steel bridge

Year	2003
Type	Rail
Country	United States
Cause	Natural hazard (Wind)
Details	A tornado, with estimated speeds exceeding 140 km/h, produced a complex pattern of high-velocity winds that attacked the viaduct
Fatalities	0
Injuries	0
Collapse	complete
Phase	in service
Reference	LEECH, T., The collapse of the Kinzua Viaduct, American Scientist, Vol. 93, No. 4, 2005, pp. 348-353.

Figure 54

Figure 55

Imola Avenue Bridge, Napa, California

Year 2003

Type Road

Country United States

Cause Human error

Details Three 100-ton hydraulic jacks used to raise the project's falsework, constructed to support the poured-in-place concrete bridge deck, were placed up to 2 inches off-center, predisposing them to shifting under the weight they supported.

Fatalities 1

Injuries 7

Collapse partial

Phase construction

References http://enr.construction.com/news/transportation/archives/031205.asp
http://www.kfty.com/news/local/story.aspx?content_id=991A9D5B-6CCA-439C-B805-A0F1B28B7C6F

Figure 56

OSHA's investigation uncovered evidence that the three 100-ton hydraulic jacks used to raise the bridge construction's falsework were placed up to two inches off center, predisposing them to shifting under the weight they supported.

Interstate 95 Bridge in Bridgeport, Connecticut

Year 2004

Type Road

Country United States

Cause Human error

Details Car collided with a 36'000 l home heating oil tanker. After the ignition, fuel oil that had dropped through a drain onto a local road, also began to burn, partially melting steel girders holding up I-95. The road dropped 1.2 m before firefighters stabilised the steel's temperature with water.

Fatalities 0

Injuries 0

Collapse partial

Phase in service

Reference http://www.newsday.com/news/nationworld/nation/ny-usmain273725186mar27,0,2627324.story

Figure 57

West Grove Bridge in Silver Lake, Kansas

Year	2004
Type	Rail
Country	United States
Cause	Impact
Details	Bridge collapsed as a result of the derailment of forty of the coal train's 137 cars
Fatalities	0
Injuries	0
Phase	in service
Reference	http://www.casperstartribune.net/articles/2004/03/29/news/wyoming/8211f8a24375fb6087256e660079bda2.txt

Crews worked Monday to pull 40 cars from a train transporting coal that derailed into a creek and caused a bridge to collapse.

Interstate 20 Bridge near Pecos, Texas (Salt Draw River)

Year	2004
Type	Road
Country	United States
Cause	Natural hazard (Flooding)
Details	Normally dry river swollen with floodwaters from two days of heavy rain
Fatalities	0
Injuries	0
Collapse	partial
Phase	in service
Reference	http://www.kxan.com/Global/story.asp?S=1761274&nav=0s3dM2gG

A storm system passed through the southwest U.S. and western Texas from April 3-6, 2004, resulting in multiple waves and heavy rainfall over west Texas and southeast New Mexico. The heavy rainfall led to significant flash flooding in small rivers there are not supposed to have. Flood waters rushed out of the Davis Mountains through Salt Draw towards the Pecos River resulting in the collapse of an Interstate-20 bridge ten miles west of Pecos

Figure 58

Lee Roy Selmon Expressway, Tampa Bay, Florida

Year 2004

Type Road

Country United States

Cause Natural hazard (Flooding)

Details Sinkhole developed under a concrete pier causing the bridge to drop 4.5 m, and the elevated roadway being built on top of it sagged, causing the bridge to buckle and collapse.

Fatalities 0

Injuries 0

Collapse partial

Phase construction

Reference http://news.tbo.com/news/MGAU1Q0J0TD.html

Figure 59

Bridge near Pawnee City, Nebraska

Year 2004

Type Road

Country United States

Cause Design error

Details Failure of falsework, bridge collapsed during concrete pouring

Fatalities 0

Injuries 3

Collapse partial

Phase construction

Reference http://www.beatricedailysun.com/articles/2004/04/23/news/news1.txt

The south end of the bridge was poured and fresh concrete was poured for an amount of 110 yards, pumped in the middle span of the bridge at the time of the accident, when it was planned of 190 yards of concrete for all the bridge.

Probably the falsework design, that it is the responsibility of the contractor on a bridge of this size, is the cause of the failure.

Figure 60

Shannon Hills Drive Bridge, Arkansas

Year 2004

Type Road

Country United States

Cause Overloading

Details Road crews were using a crane to add a pedestrian crossing on the bridge. When they finished, they drove the crane across the bridge; it collapsed from the weight from the crane.

Fatalities 0

Injuries 0

Phase in service

Reference http://www.katv.com/news/stories/0404/141620.html

Interstate 70 Bridge in Denver, Colorado

Year 2004

Type Road

Country United States

Cause Design error

Details A 40-ton girder was temporarily braced to the existing bridge with five metal bars spaced along the 30 m length. The bracings, fastened to the bridge with bolts, came loose as the girder collapsed. Girder fell on vehicle on road underneath.

Fatalities 3

Injuries 0

Collapse partial

Phase construction

Reference http://www.denverpost.com/Stories/0,1413,36~53~2151748,00.html

in 1964, when Interstate 70 through Denver was completed, the structure was designed to last 30 years. As a result, considering also that the system was payed with federal dollars, the highways were designed to meet only the minimum standard necessary for the projected traffic load to avoid having the federal government indirectly subsidize maintenance.

Figure 61

Interstate 10 Bridge, Escambia Bay, Florida

Year	2004
Type	Road
Country	United States
Cause	Natural hazard (Hurricane)
Details	Hurricane Yvan
Fatalities	0
Injuries	0
Collapse	partial
Phase	in service
Reference	http://www.e-commatrix.com/PBB/dnb-copy.html

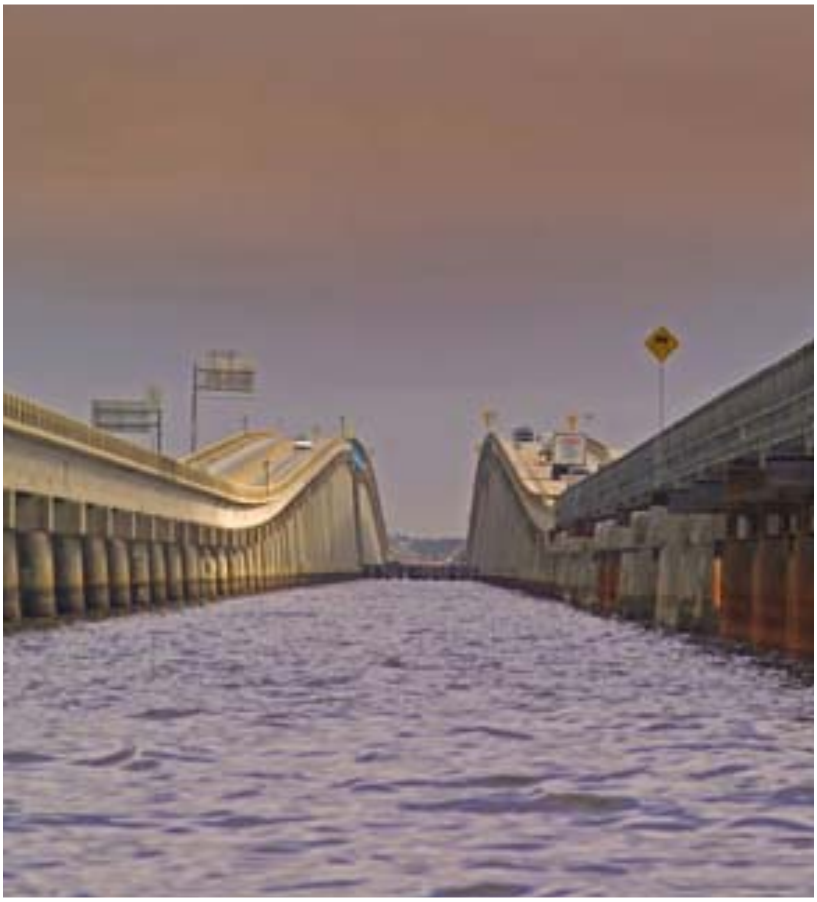


Figure 62

Figure 64-65

Hurricane Ivan struck Pensacola, Florida on September 16, 2004, with wind velocity of 120-mile-per-hour, causing severely damages like in the 2.5 mile I-10 bridge over Escambia Bay. The hurricane's damage resulted in over 3,400 feet of the bridge that drops into the bay.

Hurricane Ivan destroyed over 24 pile bridge bents.

McCormick County bridge east of Mount Carmel (Little River), South Carolina

Year 2004

Type Road

Country United States

Cause Natural hazard (Debris in water)

Details Debris from the remnants of Hurricane Jeanne stacked against the bridge's support piles in the 6 m-deep water and led to its collapse

Fatalities 0

Injuries 0

Collapse complete

Phase in service

Reference http://www.wistv.com/Global/story.asp?S=2362782&nav=0RaPRQez

http://bridgehunter.com/category/tag/through-truss/page24/

Figure 66

Bridge northwest of Norcatur (Sappa Creek), Kansas

Year	2004
Type	Road
Country	United States
Cause	Overloading
Details	Heavy grain trucks over the bridge weakened its supports
Fatalities	0
Injuries	2
Collapse	partial
Phase	in service
Reference	http://www.mccookgazette.com/story/1079462.html

Deputy Barry Richards told Herald reporters that officers assume that heavy grain trucks over the bridge weakened its supports.

The 57-foot timber bridge was rated to carry six tons on two axles, and a maximum of 13 tons on five axles.

Rural bridge near Shelby, North Carolina (Beaver Dam creek)

Year 2004

Type Road

Country United States

Cause Natural hazard (Flooding)

Details Bridge washed out. Drivers were unaware the bridge was washed out and they basically drove one after another into the swollen river.

Fatalities 1

Injuries 2

Collapse complete

Phase in service

Reference http://mobile.newsobserver.com/front/v-pda/story/1917686p-8264097c.html

The collapse leaves one man dead and injured two other people into the icy waters of Beaver Dam Creek.

Forensics haven't said yet what caused the bridges collapse. A few months before the collapse, workers had removed debris that collected under one part of the bridge. The day of the collapse a flood washed away the bridge and drivers drove in the river.

Laurel Mall Pedestrian Bridge between the parking and shopping areas

Year	2005
Type	Foot
Country	United States
Cause	Deterioration
Details	The bridge was attached by large metal bolts and brackets, which had corroded.
Fatalities	0
Injuries	0
Collapse	complete
Phase	in service
Reference	http://www.washingtonpost.com/wp-dyn/content/article/2005/07/01/AR2005070102089.html

The concrete, 50-foot long walkway in a shopping area collapse fortunately not during business hours so there were no injuries.

The bridge, which connects the second story of the parking garage to the stores, was attached by large metal bolts and brackets, which had corroded by rust.


Figure 67

Lakeview Drive Bridge, Interstate 70 in Washington County (Pennsylvania)

Year	2005
Type	Road
Country	United States
Cause	Deterioration
Details	Forty-five years of corrosive road salt draining onto one side of an overpass and a history of trucks hitting its underside likely caused a 16 m-long, 60-ton concrete beam to come crashing.
Fatalities	0
Injuries	0
Collapse	partial
Phase	in service
Reference	http://www.post-gazette.com/pg/05363/629440.stm

A guard rail and part of concrete structure fell 20 to 30 foot down of between 50 and 60 feet to the road below. The interstate seems remain intact, even though it was really cracked. There were minor injuries; just one car ran into the overpass once it had failed.

After forensic inspections, it was realized that the bridge inspections might not show the full impact of rain and salt in the corrosion of concrete and metal supports inside the beams.

In fact chloride from deicing salt percolated through the bituminous asphalt bridge surface and penetrated concrete beams.

The 45-year-old bridge also had sustained damage from oversized trucks, so precise reasons for its collapse still aren't clear and most likely involved a combination of factors, including design, weather, wear and tear.

Figure 68

Figure 69

Eight-lane, 1,950-feet-long Interstate 35-West Bridge in Minneapolis (Mississippi River)

Year 2007

Type Road

Country United States

Cause Design Error

Details Structural fatigue and the lack of a backup system in the event of a failure caused the I-35 bridge to buckle during the evening rush hour.

Fatalities 12

Injuries 0

Collapse Partial

Phase in service

References http://abcnews.go.com/US/story?id=3441034&page=1
http://en.wikinews.org/wiki/Highway_bridge_in_Minneapolis,_Minnesota,_collapses

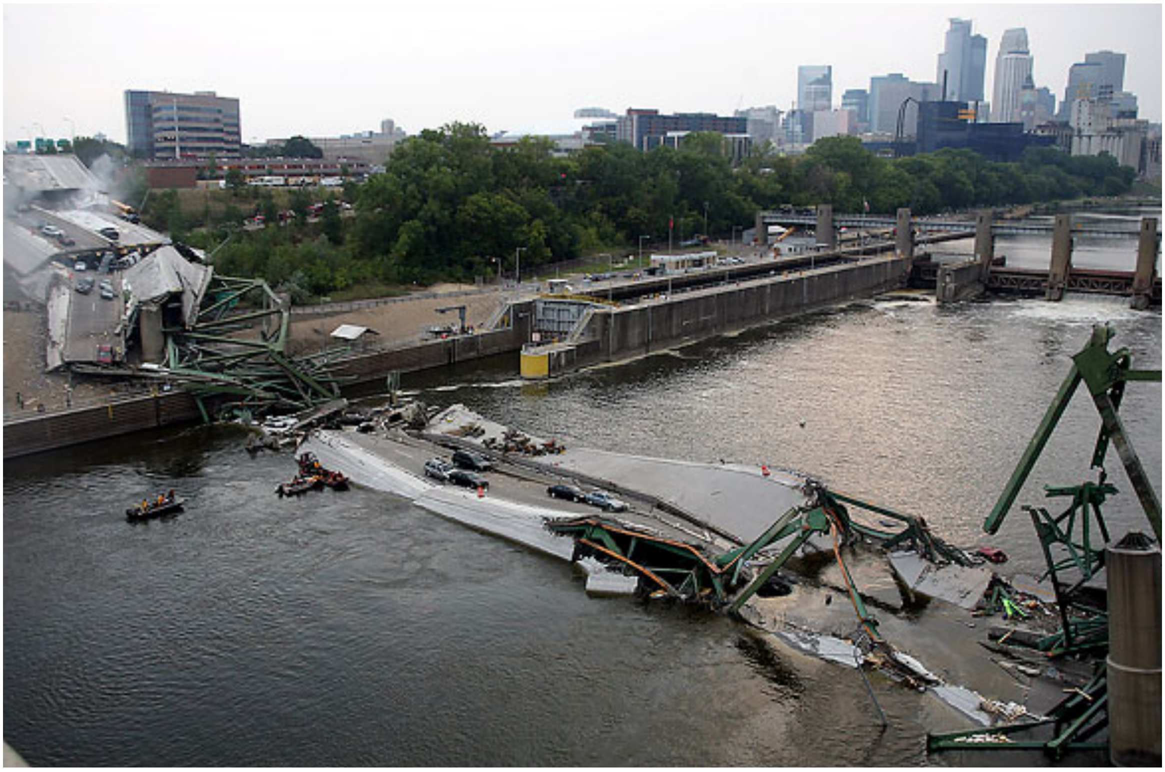


Figure 70

Figure 71

Structural fatigue, and the lack of a backup system in the event of a failure, may have been factors in the collapse of the Interstate 35-West bridge in Minnesota.

It is not still known exactly what caused the bridge to give way in Minneapolis, but state reports from 2001 and 2005 indicated there were ***fatigue cracks*** in the bridge's trusses, and that the bridge had no secondary system to bear the weight of traffic in the event of an unexpected failure. ***(no redundancy)***

The bridge "exhibited several fatigue problems, primarily due to unanticipated out-of-plane distortion of the girders. Concern about fatigue cracking in the deck truss is heightened by a lack of redundancy in the main truss system,'' a 2001 report by the Minnesota transportation department found.

"Structural fatigue and fatigue cracks" could have contributed to the collapse, Roberto Ballarini, a structural engineer and head of the civil engineering department at the University of Minnesota, said.

The 1,900 foot bridge is supported by two arching superstructures trusses.

The bridge had been classified as "structurally deficient," but that determination meant only that it needed to be maintained and not torn down, Minnesota Gov. Tim Pawlenty said in a press conference.

AGEING INFRASTRUCTURE AND MAINTENANCE ISSUES:

In the 1950's and 60's, a huge development in the infrastructure of America occurred when a massive expansion of highways and bridges were constructed to support the industrial growth of the country. The design of infrastructure at that time was very heavily focused on creation and lacked an emphasis on sustainability. Most of these bridges, referred to as "Baby Boomer Bridges", were designed to last 50 years. Now we are entering a period where the life cycles of the baby boomer bridges are reaching their ends and substantial repair and replacement is needed.

At the same time as our bridges are aging, the expansion of the industry continues to grow, increasing the loads on the already taxed infrastructure. From 1995 to 2004, annual travel on the Interstate Highway System increased by 2.8%; meanwhile, the system was only expanded by ½%. Truck travel has doubled in the past 20 years and is expected to double again by 2035.

Maintenance and *repair* of the existing bridges is required to improve the current condition of our infrastructure. Construction of new bridges is necessary to support the increasing traffic loads. Where will the funds for this work come from? In a time of economic uncertainty, how can the industry tackle the struggles associated with aging infrastructure?

AASHTO, the American Association of State Highway and Transportation Officials addressed these issues with an article titled, "Bridging the Gap". Written in response to a rise in public awareness of the aging infrastructure crises due to the Minneapolis bridge failure in 2007, the article is directed towards the public. It covers the main points of the issue, using facts and data to paint a picture the public can understand.

The age of U.S. bridges is presenting using multiple graphs and charts, emphasizing the point that 38% of all U.S. bridges are over 40 years old. A similar presentation is used to depict the condition of the bridges across the nation, with a total 25.4% of bridges that are structurally or functionally deficient. Compounded on the aging and deteriorating structures, the traffic loads have been increasing by 2.8% from 1995 to 2004. The impact of this congestion is explained in terms of delay caused by bottle-necking at highway interchanges. The top ten highway interchanges experience 1.5 million truck hours of delay each year. The cost of this congestion amounts to $7.8 billion dollars a year, 40% of which is caused by recurring congestion. Adding to the problem, construction costs of bridge materials have risen over 50% from 2003 to 2008. The crises is unfolded as a three-part issue of aging and deteriorating structures, increased traffic loads, and sky-rocketing construction costs at a time of economic downturn.

The solution presented in this article focuses on maintaining bridge safety and planning for the construction of the new bridges required. AASHTO calls for investment increases by all levels of government and proposed two financial options; *tolls* and *taxes*. The importance of a systematic long-term maintenance strategy is discussed with the main goal to find a balance between fixing the immediate deficit, preventing further deficit through efficient maintenance, and reducing the costs of new bridge construction. An essential part of the solution is research to advance the design of new bridges with an emphasis on sustainability.

The article of Dan M. Frangopol and Min Liu, published in "Structure and Infrastructure Engineering" in March 2007, "Maintenance and management of civil infrastructure based on condition, safety, optimization, and life-cycle cost" treats the problem of managing the maintenance of civil engineering infrastructures with respect to aging, using a theoretical approach. The article focuses on the best approaches for modeling and selecting the best maintenance plan for new and existing structures, particularly bridges.

The first objective of the article is to give a quantitative and qualitative definition of the infrastructure conditions over time. The performance indicators suggested include:

- Condition Index: the remaining load carrying capacity of a bridge is assessed by visual inspections and a rating on a previously defined scale is assigned to the structure.
- Safety Index: the ratio of the available live load capacity to the required capacity.
- Reliability Index: considering the probabilistic nature of the loadings and of the resistance of the single elements in the bridge, the probability that the loadings are higher than the resistance of the whole bridge is computed.

The writers suggest the use of Genetic Algorithms (GA) to choose the best management practice to apply on critical infrastructures. The power of these algorithms is that they can optimize the management procedures when they are subject to various conflicting constraint.

One of the constraints usually adopted in the bridge management strategies is the minimization of the Life-cycle cost. When a structure is being designed, all the future costs should be taken into account: initial construction costs are considered along with future operation, maintenance, inspection and repair costs forecasted on the useful life of the structure. The general expression for the Life-cycle cost is (Frangopol et al. 1997):

$$C_{ET} = C_T + C_{PM} + C_{INS} + C_{REP} + C_F$$

Where:

C_{ET} is the total forecasted cost of the infrastructure.

C_T is the construction cost.

C_{PM} is the expected maintenance cost.

C_{INS} is the expected inspections cost.

C_F is the expected cost to repair eventual failures of the infrastructure.

If a Life-cycle cost analysis is applied to an already existing bridge, then the cost is defined without the initial construction costs.

According to the article there are two ways to approach the management of infrastructure, specifically referring to bridges. The first is a project level approach in which maintenance is planned while considering an individual structure or a group of similar structures. The second is a network level approach in which maintenance is planned by considering the impact of a single failure of a bridge element on the whole network of transportation.

According to the first approach, Genetic Algorithms are applied on a single bridge which is considered critical in the network infrastructure, with the goal of ranking a list of maintenance actions with the following goals:

- Minimize the largest Condition Index.
- Maximize the smallest Safety Index.

- Minimize the present value of the life-cycle costs.

The second approach considers the whole network of transportation in a particular area and organizes management of aging infrastructure with the objective of improving not the performance of a single bridge but the performance of the whole transportation network. This approach also uses Genetic Algorithms considering other important indicators of the performance of the network: network reliability and cumulative long-term costs.

The network reliability can be subdivided into:

- **Connectivity reliability**: measures the probability that two nodes of the network are connected over a specific time frame.
- **Travel time reliability**: measures the probability that a trip between two nodes of the network can be made in a predetermined time frame.
- **Capacity reliability**: measures the ability of the network to maintain a required Level of service with determined traffic volumes.

The cumulative long-term costs consist of two main contributors:

- **Operator costs**: the equivalent of Life-cycle costs, including costs of maintenance and repair.
- **User costs**: a quantification of potential delays caused by bridge failures or maintenance.

Thus, according to the Network level maintenance planning, the goal is to distribute the resources over a number of bridges, giving the priority to the ones which are most critical to network performance. This process has the following constraints:

- Minimize the cumulative long-term costs over a predetermined analysis period.
- Maximize the network reliability according to the three indexes previously defined.

The article of Frangopol and Liu addresses the problems of optimizing management procedures by proposing two ways of facing the problem; at project level, considering only one bridge at a time, or at a network level, considering the whole transportation network in the area as a whole. It also gives definitions and tools to quantify the variables which will be used in such an analysis. However, the writers don't compare the validity of the two approaches by comparing them with a real case. They propose one numerical example for each of the two approaches but they don't show evidence of real cases where these processes were applied. In order to validate the models and procedures proposed in the article, the two approaches should be applied to a real bridge management case, considering also the possibility of integrating the two approaches in one single way of treating the problem of best allocating the resources. In a real case, both procedures would be needed since the funds should be first allocated to the most critical bridges of the network (using a Network level analysis) and then for each single critical bridge, a project level analysis should be carried on in order to better optimize the resources allocated. The combination of both approaches is not considered in the article.

The approaches to the problem of aging infrastructure of the two articles are evidently different. The AASHTO article reports the current state of the bridge infrastructure in the United States and underlines the problems that an aging infrastructure can cause not only to the transportation system but also to the whole economy. The article reports the actual values of investments made by some states over the past years and the estimates of the needed funds to bring the current bridge infrastructure to acceptable condition levels. The article emphasizes the fact that the funds available to each state are not sufficient to cover all the needed repairs and that a strategic optimization of the available resources is needed. However, a detailed analysis on the possible solutions to efficiently revive and monitor the aging of the bridge infrastructure is not provided.

The article of Frangopol and Liu is more focused on the analysis on the problem of managing an aging infrastructure through a rational and analytical approach. The article proposes definitions and parameters to evaluate the conditions of a single element both on its own and inside the infrastructure network. Moreover, it provides two ways of proceeding to better allocate the available resources on the most critical points of the infrastructure. However, the article does not apply the procedures to real cases and it doesn't take into account the possibility of unavailable funds.

Both articles agree on the fact that the current state of the bridge infrastructure is below acceptable standards and that there is an urgent need of a more rational and analytical way of allocating the resources.

5. CONSIDERATIONS

As written and explained more in details in the first paragraph, I summarize the principal changes in code due to the principal bridge failures as follow:

Takoma narrow bridge (1940)	
Overturning forces	1969
Aerolsticity	1996

Missisipi in Chester (1944)	
Excessive uplifting due to wind load	1969

Silver bridge (1967)	
Fatique	1996
Fatique limit state	1998

San Francisco Bay bridge(1989)	
Superstructure supports liking	1996
Ground liquefaction	1996

Cypress Freeway, Oackland (1989)	
Link in columns reinforcement	1996

All of the analyzed failures bring to new concepts and design philosophy:

- From ADS to LRFD (1935 to 1998) from a deterministic to a probabilistic approach
  - Strength
  - Serviceability
- Reliability design philosophy
  - Robustness
  - Redundancy
- Uncertainty (material and structural behavior) due to unexpected non-linear behavior and a sort of ignorance due to the randomness of many factors (1998 β factor)
- Earthquake: From a force based demand, to a displacement based capacity.

I believe that a couple of good question that an engineer has to think about before enter in the practice of structural design could be:

1. Does the increase in vehicular live load proposed by the AASHTO LRFD specification lead to stronger and stiffer composite bridges, and decrease the likelihood that deflections will control the design?
2. How do the new distribution factors, taken together with the new AASHTO LRFD loadings influence the eventual girder design moments and shears?
3. Is there a type of structural system that is more suitable for the design that can guarantee redundancy and strength, avoiding most of the uncertainty problems?

So, in this study it is point out that the bridge failures are due to a mix of causes, but the principal one are

1. Pour Maintenance, Inspection and bridge assessment
2. Design error/conceptual error/ ignorance
3. Misuse/overload
4. Pour construction
5. Human error
6. Extreme event
    - Hurricane
    - Flooding/debris
    - Earthquake
    - Impact
    - Fire

Maintenance and inspection are big issues, considering the ageing infrastructure problem that there is in the US as explained before in the comparison of the two articles. Moreover big and important decision should be taken if it is better to maintain the bridge or to replace the bridge, considering not only the cost of the bridge itself but also the impact of the community during a replacement of a bridge, that can be very difficult in particular in a very dense urban area as New York City. In the codes the problem of the maintenance, in terms of reducing cracks and considered in the design point of view from the fatigue tables and in the detailing of the principal connections. These tables are “new” in the codes and the fatigue problems, and so also the relative maintenance after few years can be addressed.

The *pour construction* as well *human error* is difficult to take into account from a design point of view. Pour construction is considered in the codes with required construction inspection and human error can be addressed during the design phase, from a practical point of view, thinking about what are the possible human error and trying to avoid these.

The overload due to inadequate use of the bridge is difficult to predict in the design phase, but if the bridge is used with the design load limits, I believe that the actual loads are in magnitude enough. Moreover the use of safety factors and conservative design can address this issue; very often the design loads are too conservative from a magnitude point of view.

On the other hand, what can be improved, is not the magnitude of the loads but their application from a dynamic point of view. Dynamic load effect are considered in the code only like a percentage of other static loads, instead to perform more accurate analysis.

The dynamic point of view it's more interesting because it is going to delineate clearly the reasons of the better general answer of using a system instead the other. The analysis of the dynamic effects in the structure is divided, as usually is done, between wind dynamic effects, seismic.

Dealing with the dynamic effect of the wind there are two different phenomena that should be inquired separately. Basically when the wind impact a structure the wind has a natural frequency that could be in the range of the natural frequency of vibration of the wind. When this happens the structure shows an uncontrolled displacement that most likely lead to failure. Supposing that the wind natural frequency is in the range of 4-6 second, this kind of problem is related more to large span bridges that can have this kind of range in its natural frequency. Wind is a complex phenomena that lead to several effect into the structure. For simplicity its effects are divided in different separate analysis even if they occur all together.

The *buffeting analysis* of bridge structures considers both, the aero-elastic behavior of the structures and the wind loading correlation. It is well-known that the wind profile is characterized by the mean velocity and fluctuation due to turbulence. Generally the stochastic nature of wind loading in the space and time is not accounted in the analysis. Buffeting analysis is a trial to simulate the fluctuation of the wind from the mean value putting 25% of the wind load as amplitude of a cosine function to represent these fluctuations. Considering the wind load parallel to the deck it is possible to erase the randomness of the wind into the space.

Flutter is a potentially dangerous vibration. The aerodynamic forces on a bridge, which are in nearly same natural mode of vibration of the bridge, cause periodic motion. Flutter occurs on bridges so that a positive feedback occurs between the aerodynamic forces and natural vibration of the bridge. In essence, the vibration movements of the bridge increase the aerodynamic load which in turns cause further movement of the bridge. This kind of phenomena occurs in both structures and in term of displacement the comparison between the two structures affirms again what it is said before for buffeting that having a stiffer structure reduces the displacement.

When wind flows around a bridge, it would be slowed down when in contact with its surface and forms boundary layer. At some location, this boundary layer tends to separate from the bridge body owing to excessive curvature. This results in the formation of vortex, which revises the pressure distribution over the bridge surface.

The vortex formed may not be symmetric about the bridge body and different lifting forces are formed around the body. As a result, the motion of bridge body subject to these vortexes shall be transverse when compared with the incoming wind flow. It is clear that transverse forces not completely symmetric cause torque in the deck. This phenomena was shown in the Takoma narrow bridge looking to the flutter instability. As the frequency of vortex shedding approaches the natural frequencies of the bridges, resonant vibrations often occur, the amplitude of which depends on the damping in the system and the motion of the wind relative to the bridges.

For *extreme event* is the same. With wind tunnel test we can perform very accurate analysis with a relative low cost, where the wind loads magnitudes are taken from a probabilistic approach with a certain return period and so I believe that this type of analysis is enough reliable. On the other hand, from analysis without wind tunnel there lackness in terms of dynamic loads and time history (that on the other hand are required in other country codes, for example in the Canadian), let's think about the buffeting analysis and flutter instabilit that it is not mentioned in all the codes that I have analyzed before.

From the earthquake load I believe that response spectra analysis is enough safe. If we consider actual earthquake and actual earthquake modified to get exactly the period of the structure, we get more or less the same results, with the response spectra analysis a little bit more conservative. In term of seismic analysis it is well-known that for long span bridges is not a compelling problem due to the fact that the natural frequencies of an earthquake and a large span bridge are far away. Avoiding the dynamic effect of the resonance, the problem it could be tough as a static problem with the external force proportional to the weight of the system mostly related to the weight of the deck. Dealing with small span bridges it should be taken into account the dynamic effect because of the natural frequency of the structure is closer to the range of earthquake frequency.

I believe that most of the problems can be solved by the sensibility and knowledge in the engineering design. Let's think about the impact in piers, avoidable many times with a different design, or the choice of different structural system, not only in terms of cost-effectiveness but also in terms of safety and structural response.

I believe that one of the most important concept that can include the solution of many issues is the *redundancy* in different design types. Let's think about network structure, or better arch network structure, in terms of redundancy, loss of one cable and impact on one cable, or simply a replacement of one cable.

This concept of network structure is further developed in web-net arch structure, where the redundancy is so high that is an impact occur in one of the "net cables" there is a perfect redistribution of the loads like a full plane section.

The study continue showing these results and comparison for web-net arch and network arch bridge with also dynamic loads. Web-net bridges are those bridges that instead of support the deck by the mean of a cables, they support the deck with a net composed by very thin cables or wire lumped at some point to form the typical triangulated pattern of the web net. Generally speaking the reasons why it is better to use the Web-net instead of a pattern of single cables are many from different points of view such as Redundancy, Seismic, Longevity and also phenomena related to the wind such as Buffeting and Flattering as it will be explained later on.

6. CONCLUSIONS

I believe that most of the changes in codes are related to a deep study and deep understand of the *structural behavior* instead to a deep understand of what happened in a failure; structural behavior, with the development of knowledge, with the research on many phenomena, with computer analysis and wind tunnel test etc., is deeply understood now, and failures due to missing knowledge and analysis is very difficult to happen.

For the failures there is not a real interest. In my research I have found more changes related to structural behavior and not many changes in codes related to structural failures or practical problems that causes cost in terms of lives and money.

Let's think about the fire failure or the animals accidents. There were not real changes in provision or specification related to these issues. It is like considered governed by fate, but we really can design against that.

Let's also think about the attachment with the superstructure, or the debris contained in a air or water flow. For the changes in code there was only a small changes (just named) for the superstructure attachments, but for the debris, there are not real specifications that impose to the designer to design against.

I believe that this code philosophy is wrong. We don't really think enough about failures and we are concentrated more on the behavior of the structure. I believe that changes in code due to failures is very important, in particular for low experienced engineer like me, who don't experienced all the history of failure in person.

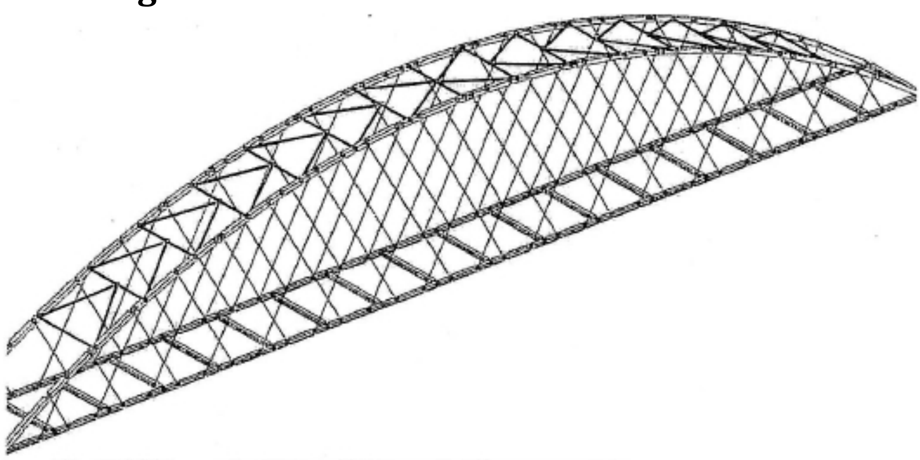
Engineers who work in the field for many years know what were the causes and try to address these problems from a practical point of view but I believe there is not the same consciousness in the codes. This is a real problem in particular for young engineers that cannot find in the codes the right answers for practical problems or for practical issues that led few years before to failures, condemned them to repeat it again.

Therefore I believe that there should be a better development related to failures in the codes and not only an awareness from a practical point of view.

7. NETWORK BRIDGES

The last step of the research is network structures usages done in the bridge field. The problem of this part is that this problematic is not almost covered by the literature and basically with the exception of the network bridge, there is no other usage. So this part of the research instead of putting some particular project that has marked the history of this kind of structure as in the rest of this work, it switch toward possible unrecorded usages of cable networks to sustain suspended bridges and cable stay bridges.

Network Arch Bridges



Reporting the definition given by Tent "Network arches are arch bridges with inclined hangers where some hangers cross other hangers at least twice. In its optimal form the tie is a concrete slab with partial longitudinal prestress.

It is best suited for small spans between 240 feet and 500 feet. This leads to attractive bridges that do not hide the landscape behind them. A network arch bridge is likely to remain the world's most slender arch bridge.

The transverse bending in the slab is usually much bigger than the longitudinal bending. Thus the main purpose of the edge beam is to accommodate the hanger forces and the longitudinal prestressing cables. The partial prestress reduces the cracks in the tie.

For load cases that relax none or only very few hangers, network arches act very much like many trusses on top of one another. They have little bending in the tie and the arches. To avoid extensive relaxation of hangers, the hangers should not be inclined too steeply. Small inclination of hangers will increase the bending moments due to concentrated loads. All hangers should have the same cross-section and nearly the same decisive load. Their upper nodes should be placed equidistantly along the arch.

Like any tied arch the network arch can be seen as a beam with a compression and a tension zone. An increased rise in the arch would give smaller axial forces in the chords and lower steel weights. It is mainly aesthetic considerations that limit the rise of the arches. Most of the shear force is taken by the vertical component of the arch force. The most part of the variation in the shear forces is taken by the hangers.

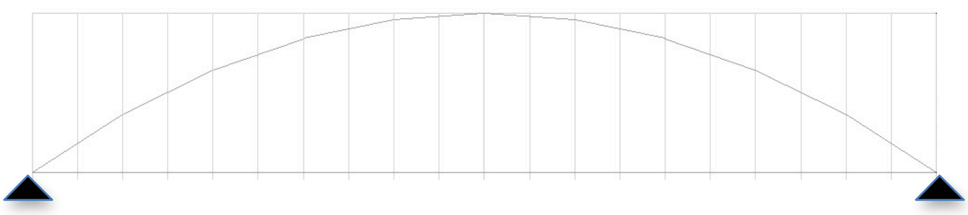
Because there is little slenderness between the nodal points of the arch, and tension is predominant in the rest of the network arch, this type of bridge makes good use of high strength steel. All members in an optimal network arch efficiently carry forces that cannot be avoided in any simply supported beam. Network arches are very stiff.

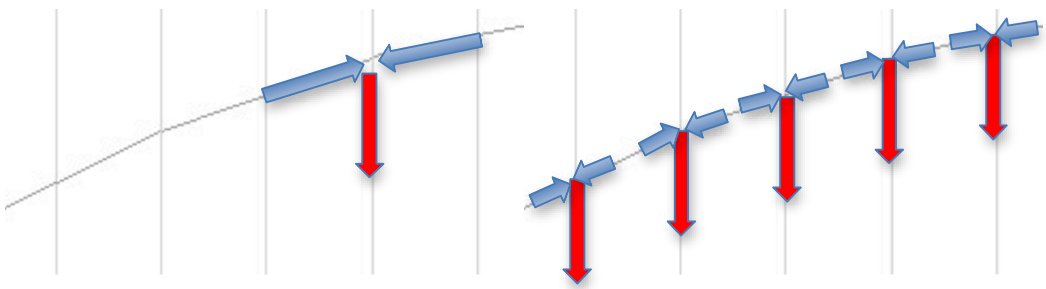
Compared with conventional bridges, the network arch, where the tie is a concrete slab, usually saves more than half the steel weight. The details are simple and highly repetitive. Thus the cost per tons is not very high. The slender tie leads to short ramps and makes it simpler to attach roads at the ends of the bridge.

The building of optimal network arches can bring great savings. However steel firms are usually not interested in using very little steel.

From the cable to the network

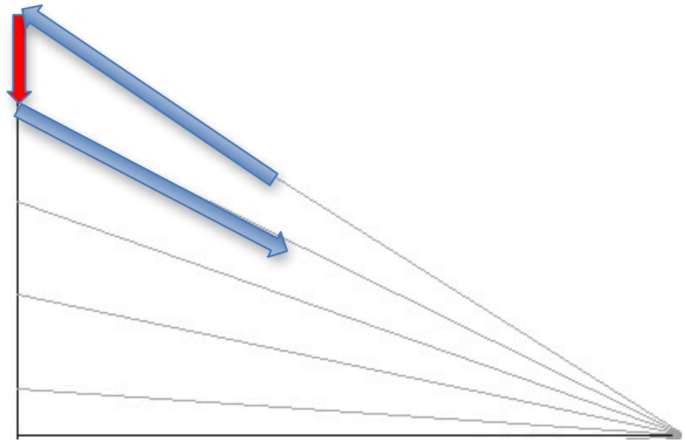
Starting from the first possible arrangement of the network arch bridge with the cable not inclined, it follows this kind of bridge in which the cables sustain the live load and the dead load of the deck transferring this load to the arch.



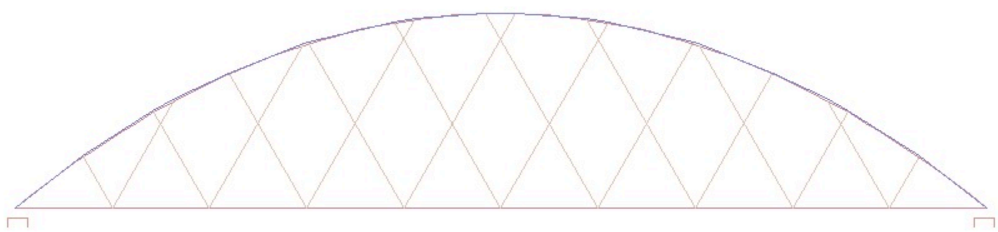


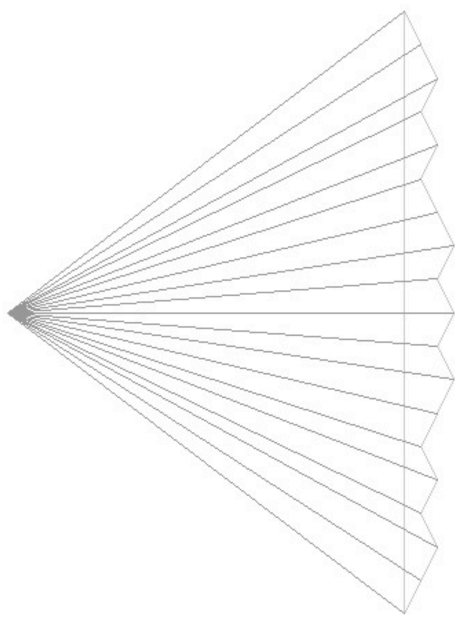
From the pictures we can easily understand how is the load path in this kind of structure and why this kind of structure is so well balanced and stiff. Almost all members are loaded in compression and tension and so as this research has pointed out yet is the best way to make structural members works as well as possible. Then as is easily understandable in the second picture part of the load transferred in the arch is mutually equilibrated by itself, between two following cables we have a component of load in the arch that is opposed and then they cancel out reciprocally.

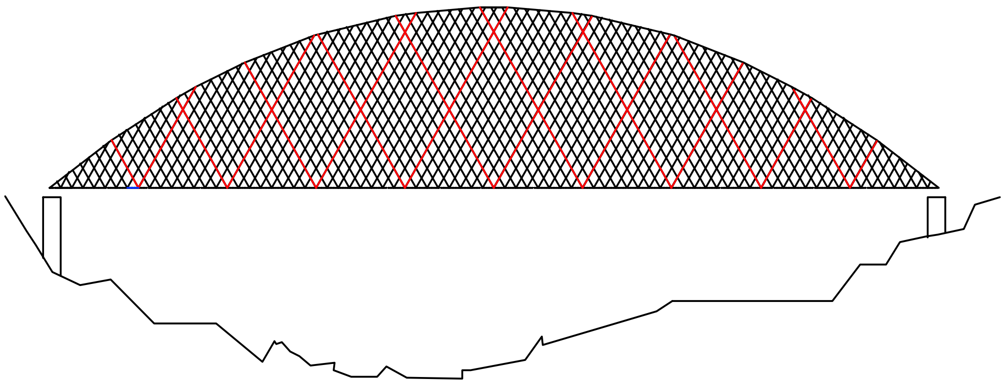
Putting the force in the cable one following the other we have the following arragment

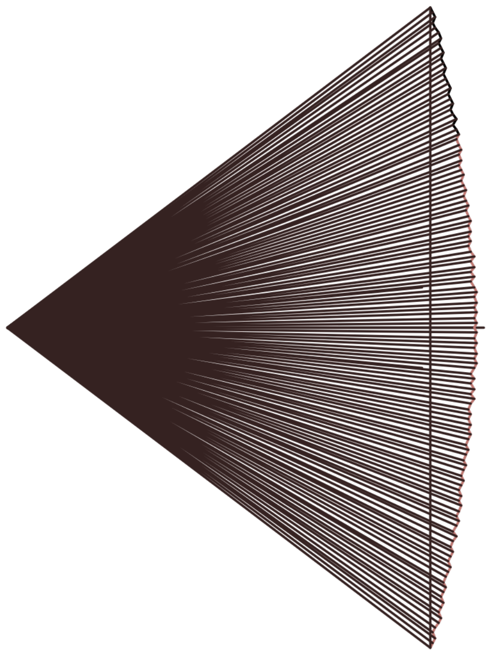


This kind of picture describes all the direction of the force and putting them in following triangles we have an equilibrated arrangementThe second step is to incline the cables and do the same thing.









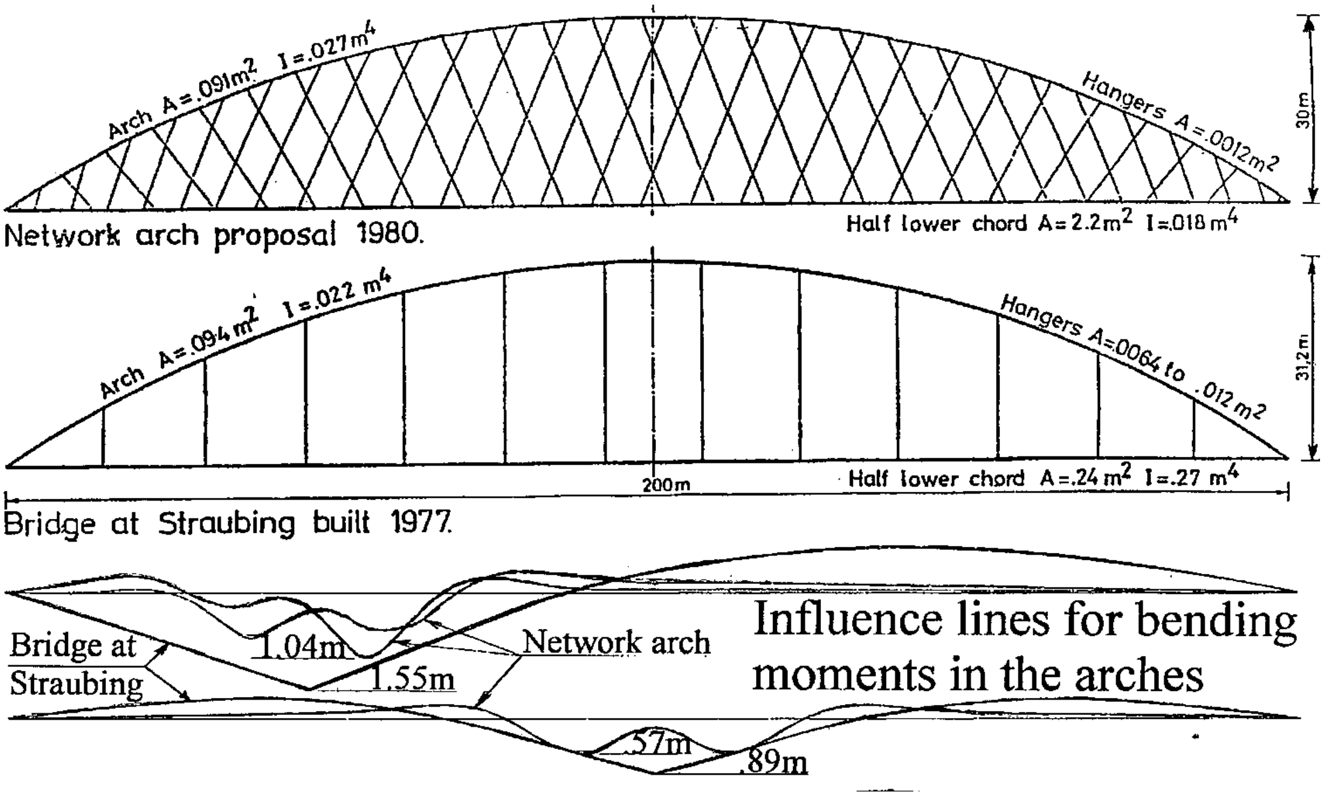
The load in the cables is inclined because follows the direction of the cable and then putting then it resultsin this kind of zig-zag arrangement on the right of the picture.

Increasing the frequencies of the load pattern we can arrive at this kind of network and do the same analisys and solve it graphically as in the other cases

The product used in this case to do the net pattern is the Jacob Inox Line Web Net.

Blennerhassett Island Network Tied Arch

In long narrow bridges, however, the longitudinal bending can become decisive mainly because much of the strength of the concrete is needed for taking the variation of the axial force in the tie. Normally a bit of extra longitudinal ribbed reinforcement is all that is needed to put things right.



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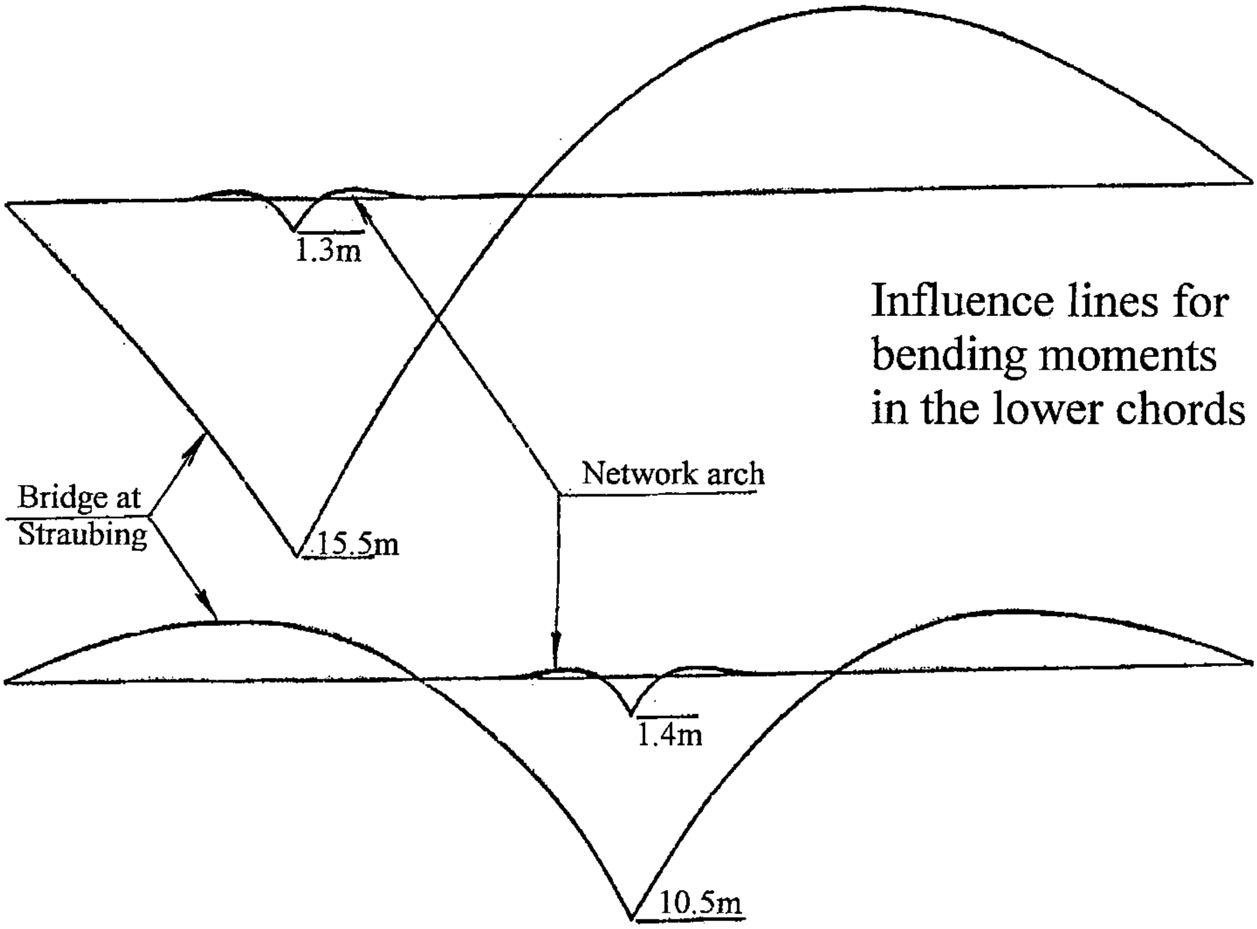
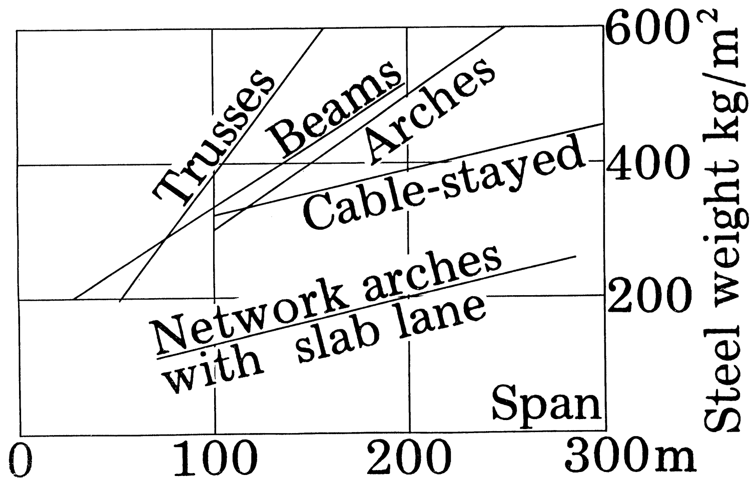
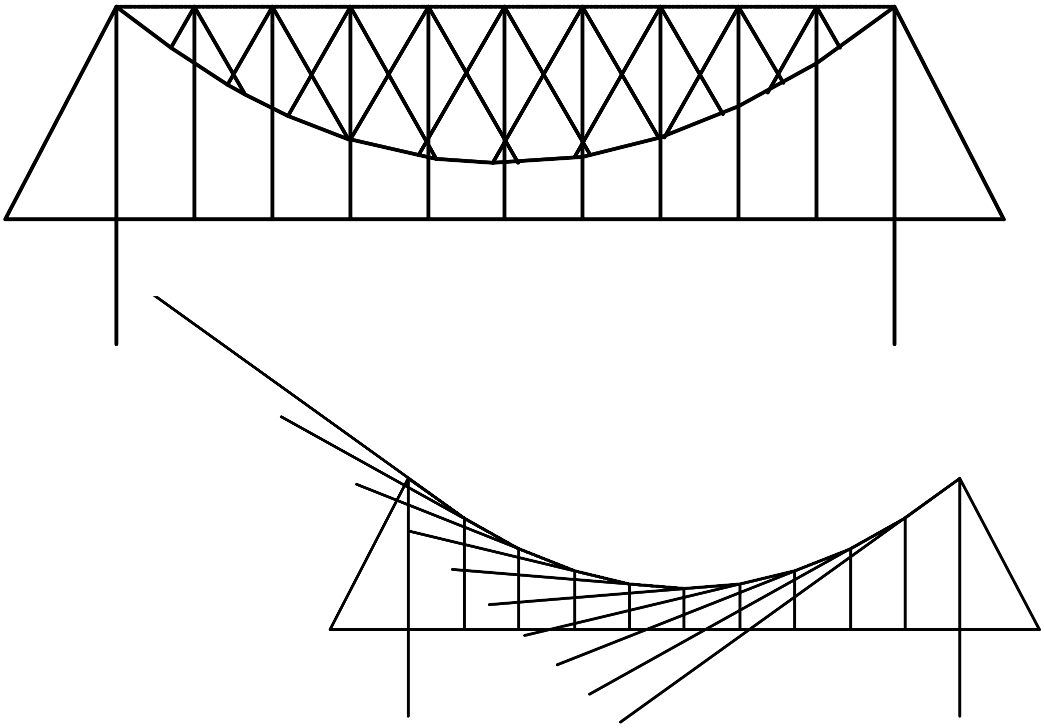

1.3m
Influence lines for
bending moments
in the lower chords
Network arch
Bridge at
Straubing
15.5m
1.4m
10.5m

Ù


Trusses
Beams
Arches
Cable-stayed
Network arches
with slab lane
Span
600
400
200
Steel weight kg/m²
0
100
200
300m

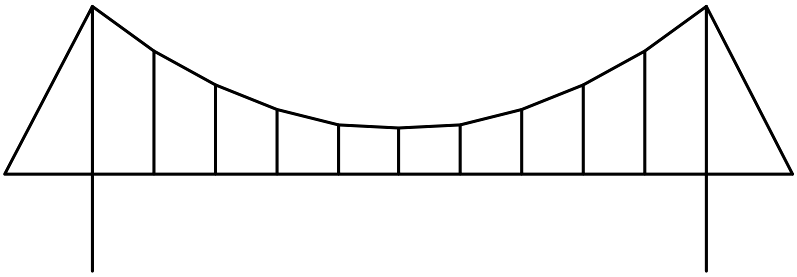
Suspended Bridge

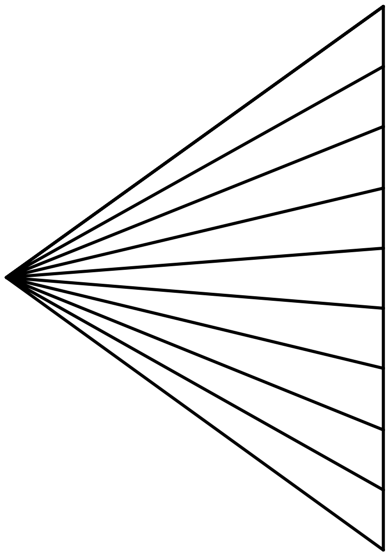


Suspension bridge has a very similar but opposed way on function of the network arch bridge.

Basically the main difference is that in this case the arch is in tension instead that in compression but the cable has always the same function to transfer the load from the deck to the main arch.

Starting from the same span of the former bridge it is possible to invert the shape of the arch passing from an arch completely in compression to an arch completely in tension according with the concept of catenary said before

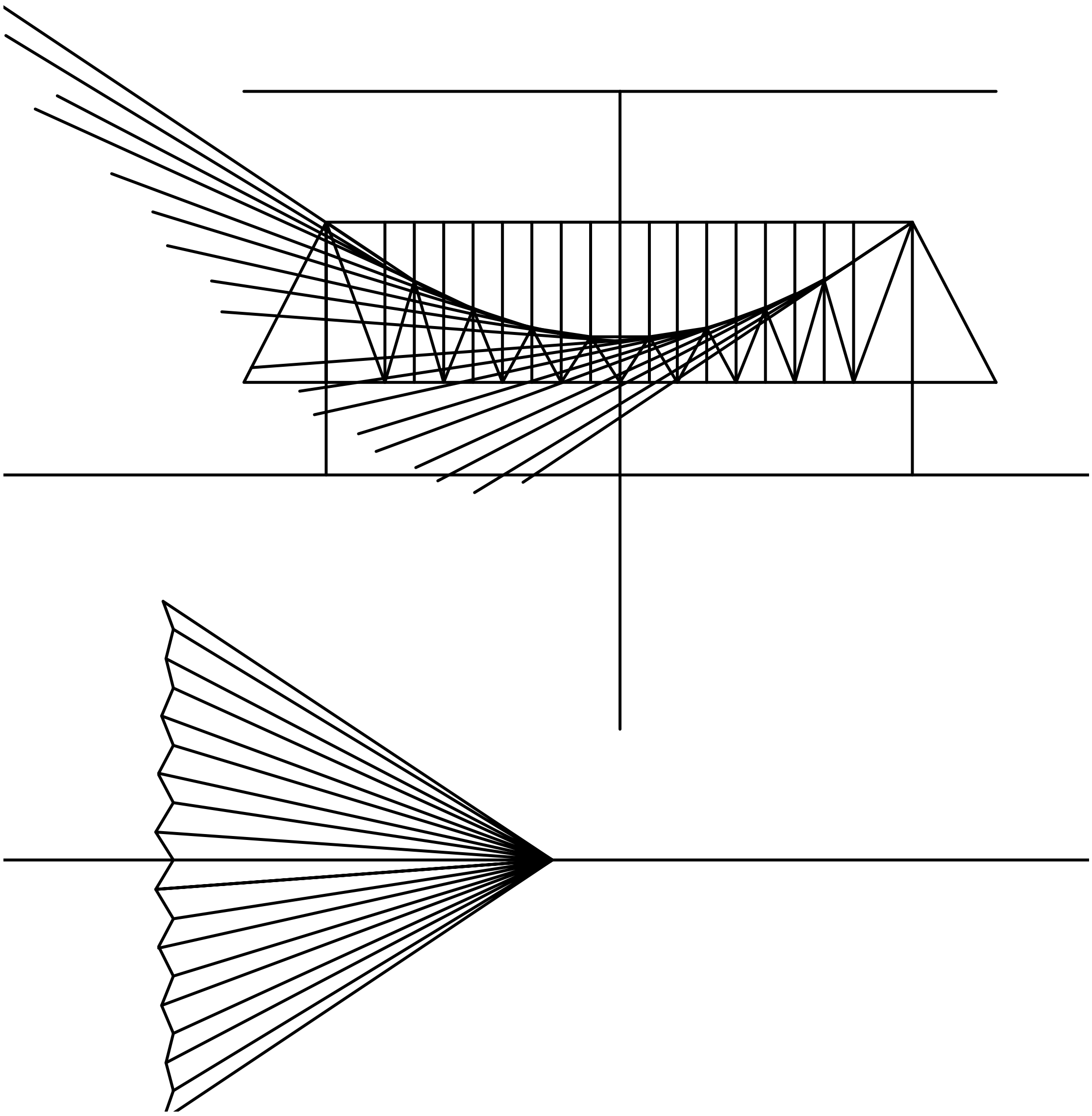




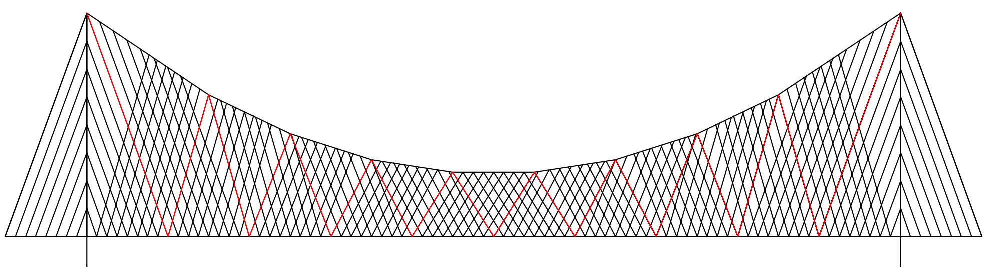
So we arrive at the same starting point of the previous bridge: a deck sustained by straight cables attached to an arch. As in the previous case it is possible to solve this structure with the help of graphic static in this lightly different because it is given the shape of the arch and we checked the equilibrium of the general system

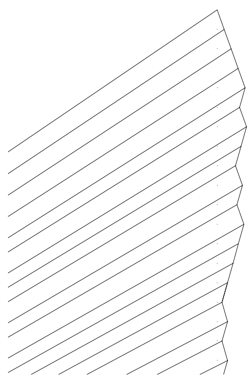
From the cable to the network

Following the same procedure of the former example we can incline the cable to have a network suspended bridge based on the bridge before doubling the cables. The system can still be solved by graphic static introducing the first step to have a Webnet Suspended Bridge

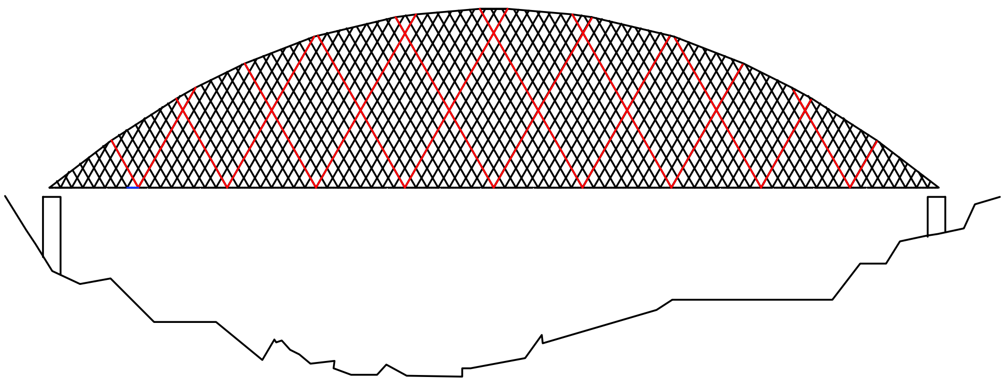


Starting from the Network suspended bridge and increasing the frequency of the cable until it is possible to reach the dimension of the typical Jacob Webnet



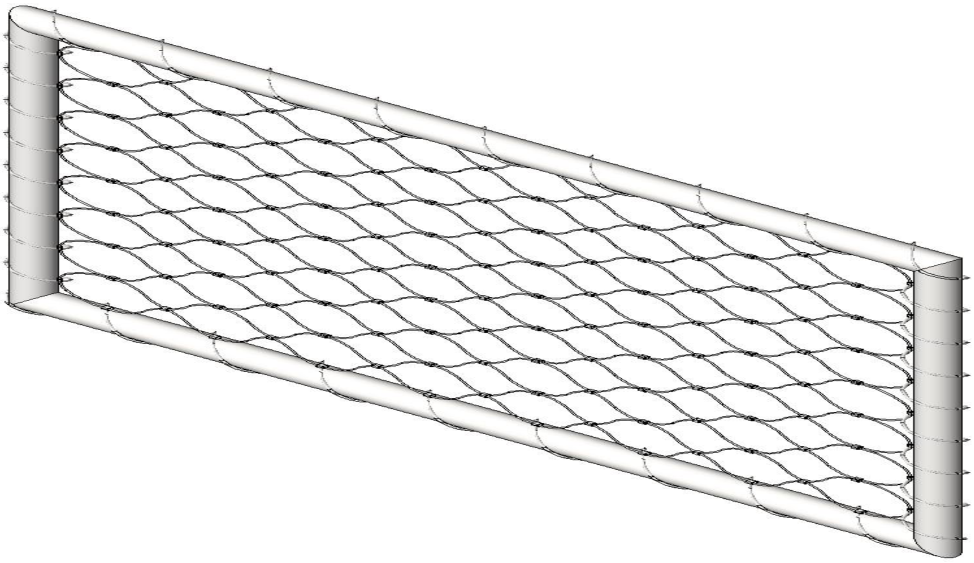


Web-net Bridge



Web-net bridges are those bridges that instead of support the deck by the mean of a cables, they support the deck with a net composed by very thin cables or wire lumped at some point to form the typical triangulated pattern of the web net. Generally speaking the reasons why it is better to use the Web-net instead of a pattern of single cables are many from different points of view such as Redundancy, Seismic, Longevity and also phenomena related to the wind such as Buffeting and Flattering as it will be explained later on.

Description



Produced by Jakob, it is a pliable, transparent grid made of stainless steel rope from Jakob Inox Line, done to be used for structural porposes and not.

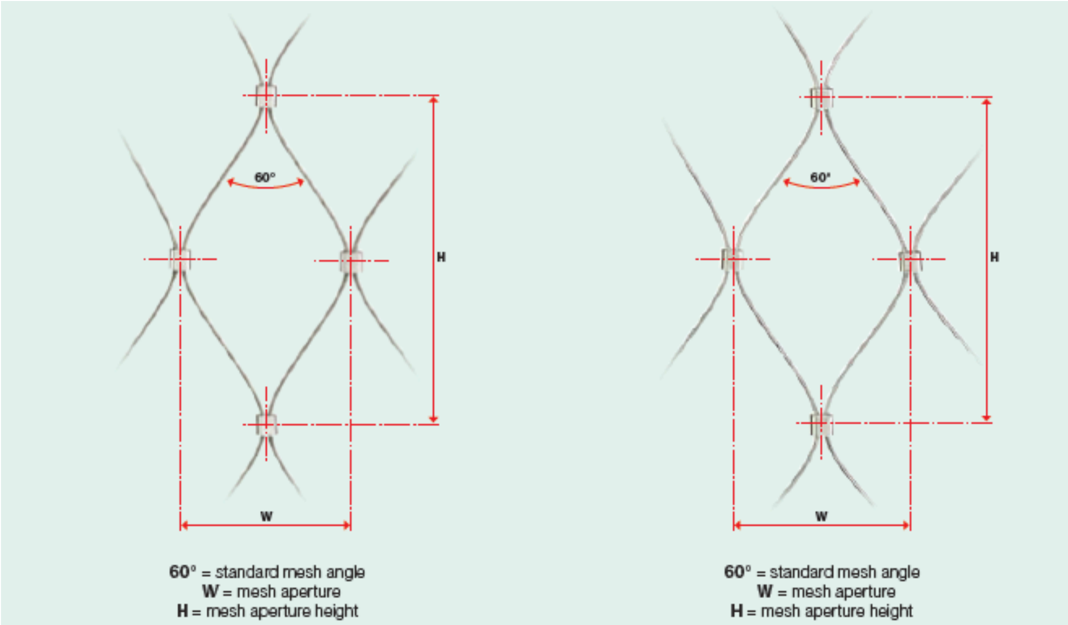
The Jakob Web-net is a permanent protective and safety net for bridges, it is absolutely UV and water resistant.

The Jacob Web-net has the skin-like characteristic of a diaphragm, it can form a plane surface or in other case be tensioned in three-dimensional forms featuring tunnel-type, cylindrical or other curved shapes.

Basically it is possible to think at the Web-net as a fabric of particular reisilience and flexibility, a net whose strands are clamped parallel in pairs, connected and reciprocally curved by offset thickness. In this way the general behavior of the Web-net can be explained as a spring whose stiffness depending on the mesh that is chosen and the thickness of the rope.

The mesh of Web-net is completely up to the designer in term of both length of the wire between lump and angle of the pattern.

The diameter rof the wire-rope in the market is from 1mm to 3 mm.



It is proposed his kind of suspending structure because of with Web-Net it is possible to catch up all the positive properties of network structure analyzed in building and use them for a new kind of suspension system for any type of bridge. The flexibility of this product that can be arranged in many ways with different mesh and different shapes make this product adapted, as shown before, to be used instead of cables in arch network bridge, suspension bridge and cable stayed bridge.

So this final part of the research is basically the proposal of this kind of new bridge and challenging the Web-net bridge with the a simple network arch bridge and suspension bridge NOTA with particular attention at the dynamic analysis.

Static Analysis

From a static point of view, having a bridge with cables or Web-net is basically the same. The weight of the deck, supposed to be constant, is distributed on the cables or Web-net equally. It is obvious that the load of the cable is much greater than the load in a single rope that composes the net. This allows a very small diameter of the cables of the net in the range of few mm. A consequence of having inclined cables is that the weight of the deck is decomposed in two forces one that goes into the cables and one that goes into the deck as compression. Thinking about compression it is important to think about flexural buckling. In this case the importance of having a net is that it reduces dramatically the inflection length having very close cables so that there is no chance to have buckling problem. If someone can argue that in network arch bridge the deck is in tension and so the compression is canceled out, the problem is the same in the arch that is in compression and so the same benefit effect of the net is transferred in the arch. In Suspension Bridges the deck is actually in compression and so it could be problematic. The general solution in suspended arch is to stiff the deck with lateral bracing increasing also the weight of the deck and so force in the cable and lateral area exposed to wind. On the other hand using the Webnet we have no problem of buckling and generally it is possible to use a light-weight deck reducing also the exposure to the wind.

One of the draw-back of having a net instead of single cables could be recognized in the static effect due to the wind impact. Basically increasing the number of the cables with a net pattern (even if the diameter of the ropes composed the net are smaller adding all the rope areas it has a net increase of 20% – 30% on cable area depending on the span) there is an increase also in the area exposed to the wind. This argument is reasonable and most likely there is this increase in the static load due to the wind, on the contrary it is possible to argue that firstly the most important answer of the structure against the wind is the dynamic effect, in particular resonance, secondly the net area increase can be cancel out by the usage of a lightweight deck without box or bracing that is just allowed by the usage of a net.

Dynamic Analysis

The dynamic point of view it's more interesting because it is going to delineate clearly the reasons of the better general answer of using a net instead of single cables. The analysis of the dynamic effects in the structure is divided, as usually is done, between wind dynamic effects, seismic.

Wind

Dealing with the dynamic effect of the wind there are two different phenomena that should be inquired separately. Basically when the wind impact a structure the wind has a natural frequency that could be in the range of the natural frequency of vibration of the wind. When this happens the structure shows an uncontrolled displacement that most likely lead to failure. Supposing that the wind natural frequency is in the range of 4-6 second, this kind of problem is related more to large span bridges that can have this kind of range in its natural frequency. Wind is a complex phenomena that lead to several effect into the structure. For simplicity its effects are divided in different separate analysis even if they occur all together.

Buffeting

The buffeting analysis of bridge structures considers both, the aero-elastic behavior of the structures and the wind loading correlation. It is well-known that the wind profile is characterized by the mean velocity and fluctuation due to turbulence. Generally the stochastic nature of wind loading in the space and time is not accounted in the analysis.

Buffeting analysis is a trial to simulate the fluctuation of the wind from the mean value putting 25% of the wind load as amplitude of a cosine function to represent these fluctuations. Considering the wind load parallel to the deck it is possible to erase the randomness of the wind into the space.

The analysis done by S.A.P and confined at the first 5 mode shapes that represent almost the 90% of the general displacement, shows an actual better answer of the Webnet Arch in term of general displacement. The better answer is due basically to the greater stiffness of the net that lead to a general stiffer bridge so less inclined to move, and a much smaller period that put the period of the structure far away from the period of resonance.

Fluttering

Flutter is a potentially dangerous vibration. The aerodynamic forces on a bridge, which are in nearly same natural mode of vibration of the bridge, cause periodic motion. Flutter occurs on bridges so that a positive feedback occurs between the aerodynamic forces and natural vibration of the bridge. In essence, the vibration movements of the bridge increase the aerodynamic load which in turns cause further movement of the bridge. This kind of phenomena occurs in both structures and in term of displacement the comparison between the two structures affirms again what it is said before for buffeting that having a stiffer structure reduces the displacement.

More interesting is a longevity analysis due to this phenomena. Assuming to have the same range of stress fluctuation do the wind, the fatigue strength depends on shape of the fracture inside the cables that it is possible to assume similar and presence of imperfections. Assuming to use the same material in the net and in cables, imperfections are more likely to occur in a big element instead of a smaller one so that the Web-net bridge answer under fatigue is much better than a single cables bridges.

Longevity is one the most important cause of collapse in bridges and so very important in designing a bridge.

Thinking about live load it is clear that this kind of load in not sustained but in transition. In this case is not critical the strength analysis that could be sufficient but not enough if it is not considered the dynamic affect due to the passage of vehicles in term of vibrations and fatigue problems due to periodic cycles of loading during the bridge life.

Vortex shedding

When wind flows around a bridge, it would be slowed down when in contact with its surface and forms boundary layer. At some location, this boundary layer tends to separate from the bridge body owing to excessive curvature. This results in the formation of vortex, which revises the pressure distribution over the bridge surface.

The vortex formed may not be symmetric about the bridge body and different lifting forces are formed around the body. As a result, the motion of bridge body subject to these vortexes shall be transverse when compared with the incoming wind flow. It is clear that transverse forces not completely symmetric cause torque in the deck. As the frequency of vortex shedding approaches the natural frequencies of the bridges, resonant vibrations often occur, the amplitude of which depends on the damping in the system and the motion of the wind relative to the bridges.

In term of torsional answer it is possible to argue that the actual improvement in suspension and network arch bridges is reached using inclined cables that resist torque by bracing the structure in the cross section plane. Straight cables do not do that. Having said that the problem to analyze is always in term of stiffness of the general system. Web-net bridges show a general greater stiffness, that allow this structure to resist better in case of torque, but more important the deck is stiffened by the net in a more homogeneous way so that avoiding local problems in the deck particularly dangerous in case of torque into the deck.

Seismic

In term of seismic analysis it is well-known that for long span bridges is not a compelling problem due to the fact that the natural frequencies of an earthquake and a large span bridge are far away. Avoiding the dynamic effect of the resonance, the problem it could be tough as a static problem with the external force proportional to the weight of the system mostly related to the weight of the deck. So in this case as just said before there is no comparison between Web-net bridges that allow a lightweight deck with network arch and suspension bridges that need a heavier deck for compression and torsion.

Dealing with small span bridges it should be taken into account the dynamic effect because of the natural frequency of the structure is closer to the range of earthquake frequency. Even if the induced force is less in the Web-net due to the less weight of the deck, the dynamic analysis is more compelling. Assuming that the frequencies of the Web-net and Network bridges are similar both system are going to get into resonance in a similar range. In this case what rules the answer of the of the system is the natural frequency of the system (m, k) and the damping of the structure. Thinking in this case at the same deck for Web-net and Network arch bridges the damping it s the same the mass of the system is the same but the stiffness is different and help the Web-net arch to have a better answer. With different decks it is more difficult to arrive at a final result because having an heavier deck means more inertia against the displacement but also a greater external force that loads the system so these two effect could cancel each other. Having a lightweight deck means a structure more inclined to move due to less inertia but loaded by smaller external force. Also in this case the different between the two system is the more stiffness and braced structure of the Webnet that generate a better structural system

Redundancy

The ultimate trend of the code is to increase the redundancy of the structures to have alternative load paths in case of unexpected failures.

There are bridge structures that are intrinsically not redundant and so they cannot provide alternative load paths. In this chapter the redundancy is dealt in particular in its relationship with progressive collapse due to cable loss.

Cable loss is generally tough as a static phenomena of having a structure without a member. Actually the cable loss is a dynamic event and it is related mostly with the duration of the event, the number of member engaged.

Focusing the attention on Arch Bridges, according with the results of the article " Some consideration in the design of long span bridges against progressive collapse", comparing a network arch bridge with vertical suspenders and inclined suspenders it is demonstrated that inclined suspenders reduced moment demands in the tie girder under plate loss and they reduced also moment demands in arch rib and enhance the stability of the former in case of cable loss.

The fundamental step is to change vertical suspenders with inclined ones to have a hyperstatic structure with the ability for load redistribution in case of single or multiple cable loss. Going further on this analysis, the more redundant the structure is the better the redistribution of the system is. For this starting from network arch bridges, it is possible to point out that if the tied girder, instead of cable with few connections each other, is supported by a net characterized by higher redundancy, the Webnet arch provides better answer also against cable loss. In the case of Webnet arch a lot of members are engaged in redistributing the extra load and both the arch rib and the tied girder shows more stability and less moment demands. In this case the Webnet arch is just the logical consequence of the passage from vertical suspender to inclined one increasing the redundancy of the former to a higher level. The Web-Net guarantees a greater redundancy and so all the positive consequences in term of alternative load paths, cable loss and progressive collapse

Conclusions

the application of network structure also in bridges where network structures are basically never used. This lack of case history and examples give me the chance to envision a different kind of suspension system that instead of using cables uses the Web-Net. After having proposed the Web-net Bridge I passed over a challenge between having a net or having a cable in Network Arch and Suspension Bridge with particular attention of dynamic effects and redundancy in case of unexpected events. After having done all the analysis what it comes out is that a net used instead of cables gives a general better answer of the structures due to the greater stiffness and so smaller displacement. Furthermore a net bracing more and in a more homogenues way the arch and the deck make these bridge less inclined to have local failure. In term of longevity there is no match between net and cables because smaller rope are actually less inclined to have fractures. Finally the more redundancy of the net make these bridge more performing in case of cable loss and progressive collapse.

This research analyze the possibilities of using network structures in all the field of constructions: buildings roofing and bridges. After this analysis it is shown that network structures could be a way to have very efficient and performance structures. Particularly interesting was the analysis about bridges, where the topic is almost not cover by the literature and case history, that lead to a new kind of suspension system for bridges that could open a new path in this field that will lead to more safe efficient and cost effective bridges.

Bibliography

T. P. Zoli, J. Steinhouse *Some Considerations in the Design of Long Span Bridges Against Progressive Collapse*

Jacob INC website, http://www.jakob-usa.com/, *Web-net catalogue*

SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2000.

SCHEER, J., Versagen von Bauwerken, Band 1: Brücken, Ernst & Sohn, Berlin, 2008.

BAILEY et al., Niveau de sécurité requis pour l'evaluation des ponts-routes existants, Report Nr. 566, Union suisse des professionnels de la route (VSS), Zurich, 2002.

Bridgeaerodynamics : proceedings of a conference held at the Institution of Civil Engineers, London, 1981.

Health monitoring of bridges [electronic resource] / Helmut Wenzel.

Bridge management / Bojidar Yanev: Hoboken, N.J. : Wiley, c2007.

Forensic structural engineering handbook [electronic resource] /RobertT.Ratay.

AASHTO 1935, 1969, 1996, 1998

Standard Specifications for Highway Bridges, 17th Edition -2002

Allowable stress design in piles

AASHTO bridging the GAP

University of Wisconsin-Madison. College of Engineering thesis

Bridge forum website

Saale-Elster Viaduct,New Erfurt - Leipzig/Halle railway line,Longest viaduct in the German railway network

Universitätsklinikum Hamburg-Eppendorf New clinic complex (UKE)

Jacob INC website, http://www.jakob-usa.com/, Web-net catalogue

http://www.brueckenweb.de/n.php

http://www.englib.cornell.edu/ice/lists/historytechnology/successfailures.html

http://www.iti.nwu.edu/links/bridges/disasters.html

http://www.basedn.freeserve.co.uk/bridge.htm