Zeitschrift: IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht

Band: 12 (1984)

Artikel: Design criteria for transit guideways

Autor: Grouni, Hid N. / Nowak, Andrzej S. / Dorton, Roger A.

DOI: https://doi.org/10.5169/seals-12165

Design Criteria for Transit Guideways

Critères de dimensionnement pour voies ferrées urbaines

Bemessungskriterien für Transit-Schienenverkehrsträger

Hid N. GROUNI
Senior Res. Eng.
Min. Transp. & Commun.
Toronto, ON, Canada

Andrzej S. NOWAK
Assoc. Prof.
Univ. of Michigan
Ann Arbor, MI, USA

Roger A. DORTON
Manager, Struct. Office
Min. Transp. & Commun.
Toronto, ON, Canada

Hid Grouni received his MS from Purdue University in 1960 and has worked in Canada, the US and the Middle East. With the Ministry since 1970, he has been involved in the preparation of the Ontario Bridge Design Code and research and design of highway bridges and transit guideways. He is a member of the ACI Bridge and Guideway Committees.

Andrzej Nowak has been on the university staff since 1979. He received his PhD in 1975 from the Warsaw Technical University. He is also involved in structural reliability, seismic risk evaluation, modeling human error and code optimization. He is a member of ACI Bridge, Guideway and Safety Committees.

Roger Dorton graduated in Civil Engineering from the University of Nottingham and received his PhD in 1954. He was a consulting engineer before joining MTC in 1972. He is chairman of the committee responsible for the Ontario Bridge Design Code. He is a member of various AASHTO, ACI and CSCE committees. Presently he is manager of the Structural Office.

SUMMARY

The paper describes the development and calibration of the design criteria for transit guideways being built in the Toronto region as part of the GO-ALRT System. The criteria are based on the limit states design philosophy and are applicable to structural steel and prestressed concrete guideways. Particular transit concerns covered in detail include vehicle-structure dynamic interaction, derailment loads, rail thermal effects, broken rail forces and fatigue.

RESUME

La contribution traite du développement des critères de dimensionnement pour les voies ferrées urbaines construites actuellement dans la région de Toronto. Ces critères sont basés sur la philosophie des états limites et sont applicables aux structures en acier et en béton précontraint. Les problèmes particuliers concernent l'interaction dynamique véhicule-structure, les effets de déraillement, de rupture de rail, de fatigue et de la température.

ZUSAMMENFASSUNG

Der Beitrag beschreibt die Entwicklung und Eichung der Bemessungskriterien für die Schienenverkehrsträger, die ein Teil des GO-ALRT-Systems in der Region von Toronto darstellen. Die Kriterien stützen sich auf das Konzept der Bemessung auf Grenzwerte ab und sind auf Stahl- und vorgespannte Betonträger anwendbar. Besondere Vorschriften beinhalten die dynamische Interaktion Struktur-Fahrzeug, die Entgleisungslasten, die thermischen Einwirkungen auf die Schienen, unterbrochene Schienenkräfte und Ermüdung.

1. INTRODUCTION

In the past, elevated transit guideways have generally been designed using provisions from existing highway and railway bridge codes in North America. However, the differences between these types of structures are significant not only in terms of values and variations in the imposed load components but also in terms on the consequences on failure of each structure. The application or the adaptation of highway or railway bridge codes to transit guideways results in rather uneconomical designs.

With the introduction of the GO-ALRT (Government of Ontario -- Advanced Light Rail Transit) project in the Toronto region, it became essential in 1983 to develop and calibrate structural design criteria to fit the specific requirements of elevated transit guideways [1]. The limit states philosophy of the 1983 Ontario Highway Bridge Design Code (OHBDC) [2] was adopted, and the material sections of the code in structural steel, reinforced concrete and prestressed concrete were utilized as much as possible. In developing load criteria some previous work of the authors was used [3, 4, 5], along with experience from the Vancouver ALRT [6], and research from the Urban Transportation Development Corporation (UTDC) transit test track at Kingston, Ontario [7].

This paper briefly describes aspects of the work newly developed specifically for the GO-ALRT Design Criteria for Elevated Guideways document, published late in 1983. Particular consideration is given to design philosophy, vehicle-structure dynamic interaction, vehicle derailment loads, rail-structure thermal interaction, broken rail forces for continuous welded rail, and fatigue.

2. DESIGN PHILOSOPHY

The traditional approach, whereby structures were designed by the working stress method and checked by the ultimate strength method did not distinguish among members of different ductilities, nor did it account for probabilistic occurrences and intensities of loads.

The GO-ALRT document is based on limit states philosophy. A limit state may be defined as the boundary between satisfactory and unsatisfactory structural performance. In a limit states design approach, a few significant limit states are first selected from a number of potential modes of failure. Next, an acceptable safety level in terms of a reliability index is established. Finally, load and performance factors are derived as part of the calibration process.

The limit states considered are those of ultimate (ULS) and serviceability (SLS). Since limit states are associated with modes of failure, ULS pertains to the load carrying capacity of a structure or its components, and SLS to its functional capacity. The former (ULS) includes force effects due to flexure, shear, axial forces, bearing, stability, buckling and rupture; and the latter (SLS) comprises those due to cracking, deflection, vibrations, permanent deformations and fatigue.

Load and performance factors were derived to yield an overall reliability index of 4.0 compared with 3.5, selected for highway structures in the OHBDC. This reflects a probability of failure in transit guideways in the order of one tenth of that expected for highway bridges.

The load and resistance factor format adopted here was similar to that established for the OHBDC. Namely:

$$\text{Factored Load} \leq \phi R_n$$

Where, R_n = nominal load carrying capacity
and ϕ = performance factor

Factored load is the sum of the effects of various load components in a loading combination multiplied by load factors.

3. LIMIT STATES AND LOAD COMBINATIONS

Three basic load combinations are considered for each of the limit states. In all cases permanent loads, such as dead loads, prestressing effects, earth pressure and track fastener restraints are included. In the ULS case strain effects due to creep and shrinkage are treated as permanent loads. Live load comprises all its derivatives, such as its vertical, horizontal and longitudinal components together with a dynamic load allowance.

The first loading combination comprises permanent loads and crush live load. However, in the ULS case, only one of the exceptional and environmental loads that produces the maximum load effect is incorporated in the group. The former includes effects due to earthquake and collision of other vehicles with guideway columns and the latter covers wind, stream flow, support settlement and temperature effects. In the SLS case this combination is divided into four sub-groups, one of which is a non-operational condition where only permanent and environmental loads are covered, with no live load effects.

The second loading combination covers fatigue in the SLS case and an empty stationary train as live load plus wind, temperature, stream flow and support settlement effects in the ULS case. Fatigue loading for the GO-ALRT system is calibrated at 80% crush loading corresponding to six million stress cycles.

The third combination comprises only permanent loads in the SLS case, designed to control cracking during construction of post-tensioned members, and an operational phase in the ULS case, where support settlement, stream flow, derailment and broken rail effects are added to those of crush load.

In the ULS case the load factor for dead load varies between 1.2 for factory produced components to 1.4 for tie-and-ballast. The load factor for live load and its derivatives is 1.3 and that for environmental loads is 1.5. Of the exceptional loads, collision and derailment are assigned a factor of 1.3 and earthquake and broken rail that of 1.5. In SLS, a factor of 1.1 is applied to live load to cover future increases in vehicle weight. The corresponding performance factors for flexure and shear are 0.85 and 0.75, respectively.

4. VEHICLE-STRUCTURE DYNAMIC INTERACTION

The dynamics of vehicle-structure interaction is one of the least known parameters in guideway design. Due to lack of more precise information, guideway designers in North America have adopted, with modifications, the AASHTO impact expression for transit. The resulting variations in their impact factor ranged between 20% and 60% of the live load [3].

In specifying the provisions for impact, or more precisely, for dynamic load allowance (DLA) in the design criteria, the dominant effect of parameters that cause resonance between vehicle and guideway was recognized. Of these, the length of a span and its fundamental frequency, together with the speed of the vehicle crossing it were considered significant. Since the upper limit for the unsprung natural frequency of most transit vehicles is above 3.0 Hz, the fundamental frequency of a guideway span was restricted to values about 3.0 Hz to avoid resonance and, hence, dynamic amplification of stresses and strains.

The crossing frequency ratio, CF/SF, is defined as the velocity of a vehicle per unit span length, V/L, divided by the natural frequency of the span, SF. The concept of crossing frequency ratio is significant in multi-span units with high-speed vehicles. The vehicle virtually launches itself from span to span.

When this action coincides with the natural frequency of the structure, resonance will be enhanced. Although tests have shown that DLA could be as low as 0.05 [7], in the Design Criteria a lower limit of 0.18 is used for values of CF/SF up to 0.30 in simple spans and 0.55 in continuous spans. Thereafter, DLA increases at different slopes (see Figure 1).

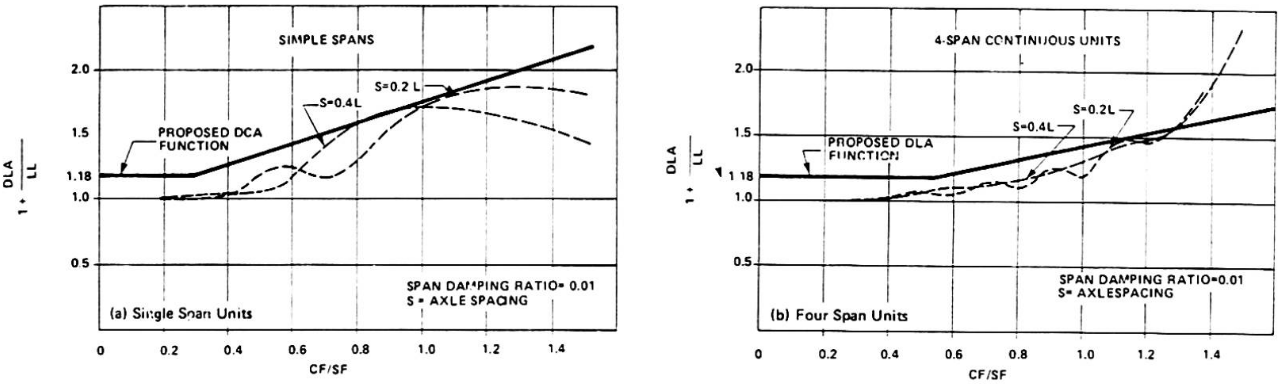

Figure 1/ Dynamic Load Allowance as a Function of the Crossing Frequency Ratio CF/SF

5 VEHICLE MISHAP

In transit systems, the emphasis on safety is much greater than in conventional modes of transportation. Extreme precautions are taken to prevent vehicles from derailing but, should they derail, their confinement within the boundaries of the guideway proper is ensured. Consequently, the barrier walls are designed to absorb all the stray kinetic energy and to confine a derailed vehicle within the guideway channel.

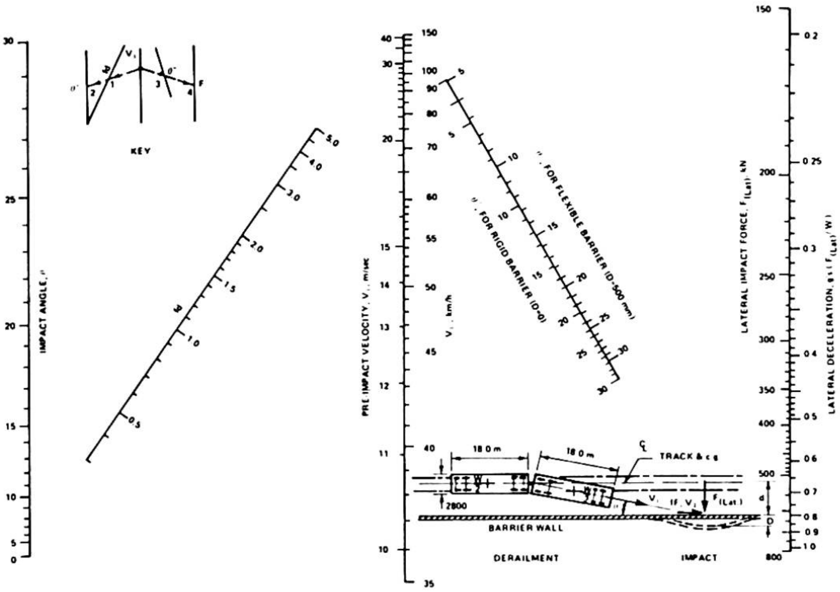

Figure 2/ Vehicle Mis-hap Force Effects

In order to evaluate more closely the magnitude of the forces a derailed vehicle might exert on a barrier wall, a nomograph was derived from test data pertaining to force effects on bridge barrier walls due to stray highway vehicles [8]. Figure 2 is a plot of these effects in terms of a GO-ALRT vehicle configuration and operating speeds, guideway dimensions and clearances, and barrier wall flexibility. The vehicle used comprises two units having a total length of 36.0 m and weight of 781 kN crush loaded with 328 passengers. The clear width of a single track guideway may be assumed to be about 2.7 m. The walls may be considered either infinitely rigid or very flexible, and a reasonable flexibility may be chosen in between. For example, using a friction factor between metal and concrete of 0.75 (fd=1) a derailed train running at a speed of 80 km/h will impact the wall at an angle of 12°, yielding a normal force of 0.47W (367 kN) on a flexible wall or 0.62W (484 kN) on a rigid wall. The resulting force may be distributed on the wall- slab system using the concept of edge-stiffened cantilever slabs [9].

6. RAIL-STRUCTURE THERMAL INTERACTION

6.1 Equilibrium Condition

With the advent of continuously welded rail (CWR) trackwork for transit, the detrimental effects of jointed rail have been virtually eliminated. These effects had an adverse contribution both to the urban environment, in the form of noise pollution, and to structural durability, in terms of increased maintenance costs. However, with the introduction of the CWR techique, some unknown factors were introduced into the analysnd behaviour of rail-structure systems, specifically in regions with high annual variations in temperature.

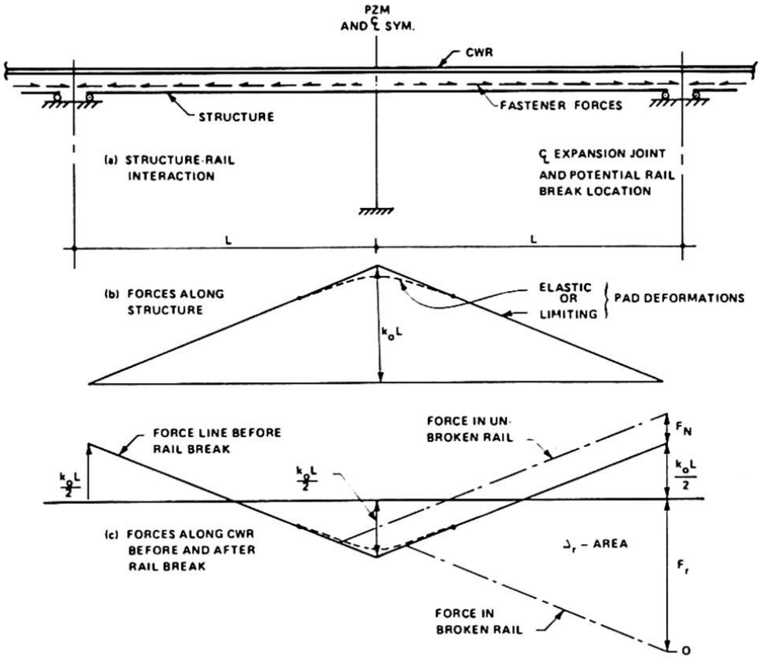

Figure 3/ Structure-rail Thermal Interaction

In a CWR track, the rail is directly attached to the deck by means of rail fasteners that comprise a clip-and-plate unit mounted on a neoprene pad. The latter possesses a measure of resilience to deform under limited amounts of guideway or rail movement. Upon reaching the limiting deformation, however, the rail slips through the clip while a constant restraining force is exerted by each pad (Figure 3a).

Since the rail is continuously restrained from movement by these fasteners, a drop in ambient temperature below that of CWR installa tion, causes a tensile force build-up in the rail; mean while, the structure contracts and moves towards the point of zero movement (PZM). In a symmetrical structure, this point is situated midway between its expansion joints; in a non-symmetrical structure, it may be found in a manner similar to that of locating the centroid of areas or forces, by using the stiffnesses of the various support elements such as piers, fasteners and bearings [10] (Figure 3a).

As the structure moves incrementally towards the PZM, some pads reach their limiting deformation whereupon the structure slips relative to the CWR. Meanwhile, the force in the rail at the structure expansion joints increases to a maximum, while at the PZM it drops to a minimum (Figure 3c). In a two-span symmetrical unit, the maximum (+) and the minimum (-) forces in the rail at the expansion joints and the PZM, respectively, may be expressed as (Figure 3c):

$$F_{r(m)} = F_r \left[1 \pm \frac{k_o L}{2F_r}\right]$$

where, $F_r = \alpha_r \, \Delta T_r \, E_r \, A_r$

and k_o is the fastener restraint force uniformly distributed along the span, L. The corresponding force induced on the structure is triangular in shape with a maximum value of k_oL at the PZM and zero at the expansion joints (Figure 3b).

6.2 CWR Broken Rail Forces

As the ambient temperature drops below that of rail installation, the probability of a rail break taking place increases. The most likely location for a rail break is at the structure's expansion joints, because there the force in the rail is highest and the rail undergoes the highest fatigue stress cycles. Once the rail breaks, its segments to either side of the joint slip through the fasteners up to the point where the cumulative fastener shear forces completely resist the net thermal stress in the rail.

As a consequence of a rail break two force effects take place: a pull-apart gap and unbalanced forces. The magnitude of the pull-apart gap is a function of many variables such as stiffness of the pads and the substructure elements, size of rail, number of tracks on the guideway, and the drop in the ambient temperature. For a two-span symmetrical structure, the rail slip or the thermal component of the pull-apart gap may be estimated from the area between the pre- and the post-break force lines (Figure 3c). Thus, the elongation to one side of the joint centreline is:

$$\Delta_r = \frac{F_{r(max)}^{\ 2}}{2k_o A_r E_r}, \text{ for stiff fasteners where, } \frac{F_r}{k_o L} < 1.5$$

and, $$\Delta_r = \frac{F_r^{\ 2}}{2k_o A_r E_r}, \text{ for flexible fasteners where, } \frac{F_r}{k_o L} > 1.5$$

The second effect of a CWR break is the introduction of an unbalanced force into the system, whose magnitude is a function of the same variables that affect the size of the pull-apart gap. In unsymmetrical structures it may be evaluated graphically as the difference between the forces at the two expansion joints of a single elevated unit. In a two-span symmetrical structure, when the new force line intersects the old one within the two-span unit, the unbalanced force is the force that existed at the next expansion joint before the break. Thus, for relatively stiff fasteners it is $F_{r(max)}$ as above, or:

$$\Delta F_1 = F_r \left[1 + \frac{k_o L}{2F_r}\right], \text{ for stiff fasteners where, } \frac{F_r}{k_o L} < 1.5$$

When these two force lines intersect beyond the next expansion joint, the unbalanced force is the total force in the fasteners accumulated between the two consecutive expansion joints at a slope of $1{:}k_o$, or:

$$\Delta F_2 = 2k_o L, \text{ for flexible fasteners where, } F_r/k_o L > 1.5$$

In this case the force transferred to the next guideway unit is the differ ence between the above unbalanced forces, or $F_{r(max)} - 2k_o L$.

The applicable unbalanced force is distributed to the support system and the unbroken rails in proportion to their relative stiffnesses. For the two-span unit considered, the share of each unbroken rail would be:

$$F_N = \frac{\Delta F}{\dfrac{K_c L}{A_r E_r} + N}$$

where N is the number of unbroken rails on the guideway, i.e., N=1 for a single track and N=3 for a double track structure. The corresponding elongation of the unbroken rails to one side of the expansion joint centreline is a function of the area shown in Figure 3c, or, $F_N L/A_r E_r$.

The force in the pier is the remainder of the unbalanced force, and the corresponding displacement may be found by dividing it by the pier stiffness. To obtain the total pull-apart gap, either the elongation of the unbroken rails or the sway of the pier should be added to the thermal component, Δ_r. In order to prevent vehicle derailment when the rail breaks, the maximum pull-apart gap in the Design Criteria was limited to 60 mm.

7. FATIGUE PROVISIONS

Little work has been done regarding the effects of high cyclic loading on transit structures. Although the stress range in the prestressing steel at the precompressed zones of the calibrated guideways [4] was well below 10% of its ultimate strength, it was found necessary to limit the tensile stress in the extreme concrete fibre to zero or possibly to a maximum of 1 MPa.

For structural steel guideways, the main fatigue parameter is the S-N (stress-cycle) curve and its distribution within the service life of the structure [11]. Since there is no predefined load distribution model for transit guideways, fatigue analysis for any system should be based on an anticipated loading spectrum. The spectrum provided by the transit agency in this case indicated that, within the anticipated six million cycles of loading, 55% were at one-half full capacity and 35% at two-thirds full capacity; the remaining 10% cycles were distributed between fully seated (5%), nominal (2.5%), and crush loads (2.5%). The vehicle weight ranged between 555 kN empty to 781 kN crush loaded with 328 passengers. The application of both Miner's and RMS rules [11] in calibration yielded almost identical results as an equivalent stress level to be used with the given load spectrum. This stress level corresponds to the weight of a GO-ALRT vehicle loaded with 100 passengers. It is equivalent to 80% crush load or 623 kN on four bogies with two axles each. Next, fatigue stress limits were stipulated in the Criteria for various structural details, as given in Reference 11. For instance, the allowable fatigue stress ranged between 124 MPa for base metal and 52 MPa for the lowest type of detail.

8. CONCLUSIONS

This has been the first attempt in North America to formulate a comprehensive set of design criteria exclusively calibrated for the design of transit guideways. Much of the material was adopted from the OHBDC [2]; it will also be incorporated in the forthcoming ACI Design Recommendations to be issued by ACI Committee 358.

The Design Criteria document has resolved problems unique to elevated transit guideways. Limit States philosophy is utilized with force effects that are imposed on guideways, such as rail-structure thermal interaction, vehicle-structure dynamic interaction, vehicle mishap loads and fatigue associated with high cycle loading. Load and performance factors were calibrated specifically for transit loads.

9. NOMENCLATURE

The following terms are not defined in the text

A_r = cross-sectional area of rail, mm^2.
E_r = modulus of elasticity of rail, MPa.
F_r = force in a rail due to a drop in ambient temperature, N.
k_o = fastener restraint force divided by fastener spacing, N/mm.
LL = live load or its effect, N.
α = thermal strain coefficient, 12×10^{-6} mm/mm/°C.
ΔT_r = drop in rail temperature below that of installation, °C.

10. ACKNOWLEDGEMENTS

The authors wish to thank the other members of the technical committee: T. Nettles of ABAM and W. McEwen, Consulting Engineer; and the contributors: C. Sadler of MTC and D. Gathard of ABAM.

11. REFERENCES

[1] MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, GO-ALRT System Standards -- Design Criteria for the GO-ALRT Elevated Guideway and Special Structures, MTC, Downsview, Ontario, Canada, 1983, p. 430.

[2] MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, Ontario Highway Bridge Design Code (OHBDC) Vol. 1 & 2, MTC, Downsview, Ont., Canada, 1983, p. 636.

[3] DORTON, R.A. and GROUNI, H.N., Review of Design Criteria in Existing Transit System Codes, ACI Journal, Proceedings V. 75, No. 4, April 1978, pp. 134-44.

[4] NOWAK, A.S. and GROUNI, H.N., Development of Design Criteria for Transit Guideways, ACI Journal, Vol. 80, No. 5, Sept.-Oct. 1980, pp. 387-395.

[5] GROUNI, H.N. and NOWAK, A.S., Safety Criteria in Calibration of the Ontario Bridge Code, International Conference on Short and Medium Span Bridges, Vol. 1 & 2, August 8-12, 1982, Toronto, Ont., Canada, pp. 183-192.

[6] METRO CANADA LTD, Greater Vancouver ALRT System Design Manual, UTDC, Kingston, Ontario, Canada, 1982, p. 520.

[7] ABAM ENGINEERS, INC., ICTS Development Program (Summary Report A 80104) Vol. 1-3, METRO CANADA LTD., Kingston, Ontario, Canada, 1981, p. 2200.

[8] NATIONAL ACADEMY OF ENGINEERING, Tentative Service Requirements for Bridge Rail Systems, NCHRP Report #86, College Station, Tx, 1970, pp. 1-58.

[9] BAKHT, B., Simplified Analysis of Edge-Stiffened Cantilever Slabs, ASCE, Structural Division, Proceedings, Vol. 107, N. ST3, March 1981, pp. 535-550.

[10] WITECKI, A.A., RINA, V.K., Distribution of Longitudinal Horizontal Forces Among Bridge Supports, First International Symposium, ACI Publications, SP-23, 1969, pp. 803-815.

[11] FISHER, J.W., Bridge Fatigue Guide -- Design and Details, American Institute of Steel Construction, 1977, p. 58.