SEMINAR

III

Transit Guideway Structures

Structures des moyens de transport en site propre

Tragwerke für Verkehrsmittel auf Eigentrasse

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Vancouver ALRT Special Guideway Structure

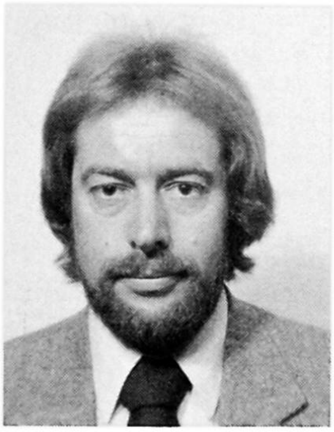
Structure spéciale pour le métro de Vancouver

Spezielle Hochbahn-Tragstruktur des ALRT-Systems in Vancouver

Robert R. PARSONS
Special Proj. Eng.
N.D. Lea & Assoc.
Vancouver, B.C., Canada

Robert R. Parsons (1942) graduated from London University, U.K., in 1965 with a Bachelor of Science Honours degree in Civil Engineering. He has a wide range of experience in both the design and construction supervision of major highway and transit bridge structures both in Canada and overseas.

Graham W. TAYLOR
Senior Design Eng.
Ker, Priestman & Assoc.
Victoria, B.C., Canada



Graham W. Taylor, born 1950, graduated from the University of Queensland, Australia, with his bachelor and master's degrees and received his doctoral degree from the University of Alberta, Canada. He has extensive experience in the design and evaluation of short to long span highway, railway, logging and transit bridges.

SUMMARY
Vancouver's Advanced Light Rapid Transit (ALRT) System is an ambitious transit project that integrates structural efficiency, environmental sensitivity, and electronic control technology. This paper describes the design and construction of a three-span continuous cast-in-place post-tensioned box girder structure that comprises a segment of the concrete elevated guideway.

RESUME
Le métro de Vancouver est un projet ambitieux qui intègre les problèmes liés aux structures, à l'environnement et au système de contrôle électrique. Cette contribution décrit le projet et l'exécution d'un caisson continu sur trois travées faisant partie du tracé surélevé.

ZUSAMMENFASSUNG
Das Advanced Light Rapid Transit (ALRT)-System ist ein ehrgeiziges Vorhaben eines Transportsystems, das bauliche Leistungsfähigkeit, Rücksicht auf die Umweltbelastung und elektronische Kontrolltechnologie vereint. Der Beitrag behandelt den Entwurf und den Bau eines an Ort betonierten, vorgespannten, dreifeldrigen Kastenträgers, der Teil der erhöhten Betontragstruktur ist.

1. INTRODUCTION

In the early 1970's the sharp increase in the price of non-renewable fossil fuels, traffic congestion, and environmental pollution in large cities stimulated keen interest in alternative and more energy efficient forms of public mass transit. The Urban Transportation Development Corporation Ltd. (UTDC) was created by the Ontario Government in 1972 to investigate and develop urban transportation alternatives. Vancouver's Advanced Light Rapid Transit (ALRT) System is the culmination of the UTDC developmental program and is the first full-scale application of this rapid transit technology. Prime contractor for the Vancouver ALRT is Metro Canada Ltd., a subsidiary of UTDC. Owner and operator of the Vancouver ALRT is B.C. Transit, a crown corporation of the Province of British Columbia.

The first phase of construction comprises a 21.4 km long dual guideway that is completely separated from all conflicting traffic to form a continuous, uninterrupted, dedicated right-of-way that will allow fast, efficient service. Thirteen kilometres of the guideway is divorced from traffic conflicts by an elevated structure that varies in height from six to eight metres above the ground. The elevated guideway superstructure is composed primarily of standardized precast concrete beam elements, the only exception being where geometric complexities dictate that a different type of construction be used. One such exception is "Special Switch Structure S232 Outbound". This latter structure is the subject of this paper which describes the design and construction of this complex segment of the concrete elevated guideway.

2. STRUCTURE LOCATION AND CONFIGURATION

Special Switch Structure S232 Outbound comprises the initial segment of the outbound elevated guideway immediately east of the Stadium Station. The special guideway structure is adjacent to the site that is being developed for Vancouver's world fair Expo '86. The special guideway structure passes over Pacific Boulevard, the main arterial route to Vancouver's domed stadium.

The Stadium Station has one storage outbound track in addition to the primary outbound track. The storage outbound track converges with the primary track east of the station and the switch is located on this special guideway structure. As a consequence, the structure is bifurcated, with the deck tapering in plan in the vicinity of the switch. The total length of the structure is 97 m.

3. DESIGN CRITERIA

Since the ALRT system incorporates specialization in the vehicles, structural system, trackwork, and communication technology, UTDC established design criteria to supersede or supplement existing design codes. The structural design was completed in accordance with the "Vancouver Advanced Light Rapid Transit System Design Manual". Aspects of the structural design not covered by the latter design manual conform to the Canadian code "Design of Highway Bridges" CAN3-S6-M78.

The design vehicle is a light automated aluminum and fibreglass car that has a normal load of 76 passengers and a crush load of 105 passengers. The live load adopted for the design is for the crush passenger load and is equivalent to 16.34 KN/m. Impact is 10% in all areas other than switch areas where the impact is increased to 30%. Each train comprises pairs of vehicles with up to six vehicles per train. The vehicle length is 12.7 m between couplers.

In addition to the considerable contribution from the girder self-weight, the superstructure dead load includes the 180 mm thick concrete deck overlay. The dead load was the dominant force in the design of the superstructure girder.

The use of continuous welded rails that are fastened directly to the deck introduces several unusual design forces and modifies the distribution of other superstructure design forces. Variations in temperature above or below the temperature at rail installation develop significant rail/structure interaction forces that must be considered in the design of both the superstructure and substructure. The presence of a switch on the special guideway structure provides an additional imbalance in rail temperature forces. The rare occurrence of a rail breaking is also considered, the loading shock being partially distributed to adjacent structures through continuity of the remaining unbroken rails. Similar force distribution to adjacent structures also occurs for localized forces such as normal or emergency braking.

In addition to rail/structure interaction forces, temperature load arises from a thermal gradient in the deck and a temperature differential between ambient air temperature and the air temperature of the superstructure's box girder void. The top surface of the deck was assumed to be 22°C higher than the underside of the top flange and underlying girder. The box girder void was assumed to be 22°C higher or 28°C lower than the ambient air temperature.

Vancouver's proximity to active seismic zones is reflected in the severe earthquake design provisions that are incorporated in the structure. The stratigraphy of the ground in the vicinity of the structure consists of a silt zone varying in depth from 3 m to 10 m overlying a stiff conglomerate till. The combination of this foundation material and the stiffness of the elevated structure resulted in a seismic design force equivalent to a static horizontal load of 20% of the vertical dead load.

4. STRUCTURAL SYSTEMS

4.1 Superstructure Structural System

The special guideway structure is illustrated in Figure 1. The superstructure consists of a post-tensioned concrete box girder supporting a sidewalk on one or both sides of the girder. The girder carries a concrete overlay, running rails and fasteners, linear induction motor reaction rail, power rails on the inside face of the parapet wall, cable conduits in a cable tray underneath the sidewalk, and an automatic train control data communication loop between the running rails.

The standard superstructure module for the elevated guideway is a two-span continuous structure comprised of two precast post-tensioned concrete box girders of span lengths up to 33 metres. Because of the non-uniform deck shape tapered in plan, the long centre span, and clearance limitations over Pacific Boulevard, a three-span continuous cast-in-place concrete box girder was adopted for the superstructure. The bifurcation of the girder to accommodate the storage track changes the box configuration from single-celled to triple-celled at the station end.

As noted previously, the rails are fastened directly to the deck. Presence of the switch requires considerable clearance underneath the rails to accommodate the switching equipment. Allowance for the switch clearance envelope led to the introduction of the 180 mm concrete overlay on top of the box girder top flange.

4.2 Substructure Structural System

The superstructure is supported by three expansion piers and a fixed pier. The expansion pier at the west end is shared with the Stadium Station superstructure. The two expansion piers east of the Stadium Station have single columns with uni-directional bearings, the columns being positioned along the girder centreline. The fixed pier has two laterally-spaced columns that are cast integrally with the girder to form a rigid connection. Adoption of one fixed pier rather than a fixed pier either side of the centre span minimized secondary stresses due to live load, post-tensioning, shrinkage, creep, and temperature change.

All pier columns are supported on piled footings, the piles being required to reduce differential settlements in the underlying silt. All piles are tubular steel piles and are filled with concrete. Footings and columns are conventionally reinforced.

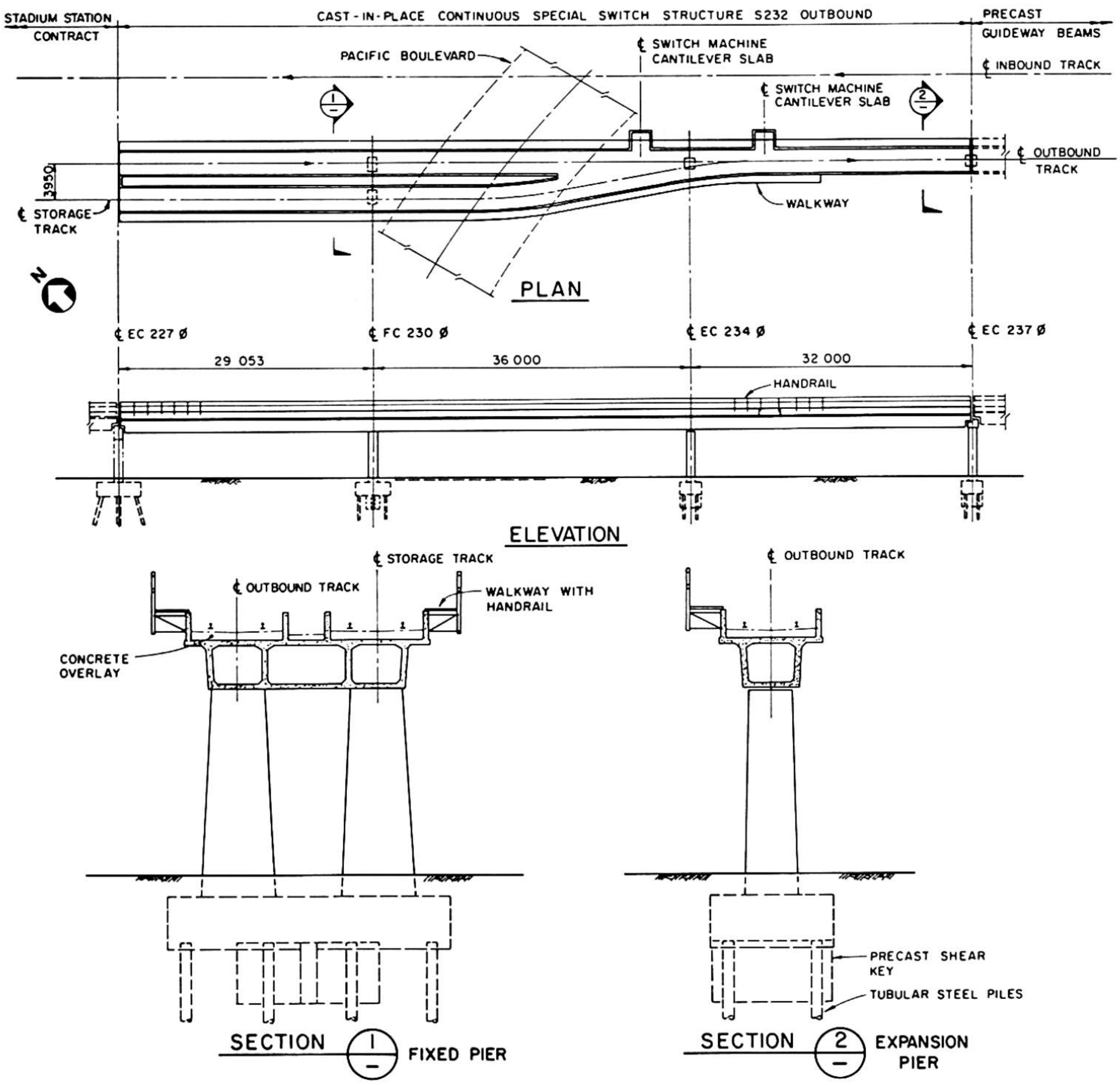


Fig. I Special Switch Structure S232 Outbound

5. SUPERSTRUCTURE DESIGN

The geometric configuration of the cast-in-place concrete box girder superstructure was determined by the alignment of the primary and storage tracks, clearance provisions over Pacific Boulevard, and adherence to the critical systemwide geometry requirements. Small cantilever extensions of the girder top flange were required to support the switch machines positioned outside the girder parapet walls.

Design of the continuous post-tensioned box girder was dominated by dead load and temperature forces. The magnitude of the live load was considerably less than the latter two forces. In addition to the principal vertical loads, secondary horizontal design forces due to rail-structure interaction and switch forces were also considered in combination with the vertical loads. The level of post-tensioning was chosen to prevent cracking in the box girder top flange but permit cracking in the parapet walls. Vertical cracking of the parapet walls is permitted as their prime function is to support the power rails and to absorb the energy from a derailed vehicle.

Design and layout of the post-tensioning was complicated by the changing box girder cross-section from three cells at the Stadium Station end to one cell at the other end. The highest level of post-tensioning was required over the fixed pier with lesser post-tensioning required at mid-span of the centre and end span adjacent to the Stadium Station. To reduce friction losses and achieve the high level of post-tensioning over the fixed pier, deck openings were provided 6.75 m either side of the fixed pier centreline to terminate and jack the tendons originating from opposite ends of the girder and crossing over the fixed pier.

As stated previously, the principal purpose of the 180 mm concrete overlay on the box girder's top flange was to accommodate the trenches beneath the running rails for the switching equipment. Since the trenches were required across the full width of the deck, the overlay was considered non-structural and expansion joints were located at regular intervals in the overlay to discourage composite action and reduce the size of overlay tension cracks over the interior pier supports.

The presence of continous parapet walls and a deck profile that slopes toward the centre of each track provides a large enclosed catchment area to be drained. The vertical alignment of the structure is a 0.5% slope toward the Stadium Station. This longitudinal slope was considered sufficient to require only one set of drains adjacent to the fixed pier. One hundred millimetre diameter PVC drain pipes have been cast in the girder and these connect with 150 mm ductile iron drain pipes at the underside of the box girder.

6. SUBSTRUCTURE DESIGN

The bent that supports the special guideway superstructure at the Stadium Station interface is not discussed as it was constructed in a separate contract. The remaining supports comprise two expansion columns and a pair of fixed columns as noted previously. Twin fixed columns are required to resist the severe longitudinal earthquake forces. In calculating column bending moments due to longitudinal forces, account was taken of the significant flexibility in the piled footings. Double uni-directional bearings were provided at the expansion column EC 237 ø to provide girder torsional fixity at the expansion joint. This measure was necessary as the excessive box girder torsional rotation at the east deck expansion joint would have developed large secondary stresses in the continuous running rail. The east end of the box girder is solid for the last 5 metres to prevent bearing uplift.

All columns are supported on rectangular, reinforced concrete footings of 1.5 m thickness. In addition to meeting the piling requirements, the footing size was influenced by the need to restrict footing movement under the action of horizontal, in-service forces. These forces included emergency braking and broken rail forces. The footing movement was considered acceptably small for the in-service condition if the lateral soil pressure did not exceed the at-rest value. Where the footing size would have become unacceptably large to meet this requirement, precast reinforced concrete shear keys were introduced at the underside of the footing slab. This increased the effective footing area in contact with the soil. The shear keys extended 1.5 m below the underside of the footing. The shear keys were precast as the groundwater elevation fluctuated considerably and was anticipated to be close to the bottom of the footing. Footing reinforcement was increased by 10% beyond that required to resist the column forces to ensure that a ductile hinge would form in the column in a failure loading condition.

The piles supporting the footing are 360 mm diameter tubular steel piles of 12.7 mm wall thickness. The pile driving specification required the piles to be driven through the 3 m to 10 m silt layer and penetrate 3 metres into the hard till. This penetration into the till provided design pile capacities of 1700 KN in compression and 180 KN in tension. Although vertical column loads were substantial, due primarily to the high dead load, horizontal design forces were significant in the design of the piling layout. The initial approach adopted was to resist all horizontal forces, including the large earthquake forces, by the use of battered piles. However, the earthquake design shears and moments required an excessive number of battered piles, the pile strength limitation being the low tensile capacity rather than the high compressive strength. In conjunction with the use of battered piles, there was also concern that the thick silt layer might experience a substantial seismic movement in the order of 75 mm. Such silt movements would generate large soil passive pressures upon encountering the relatively rigid, battered-piled footing, raising the possibility of failure in the battered piles. To resolve these two serious problems in a battered pile approach, all piles were made vertical and in-service horizontal forces were resisted by at-rest soil pressure. Under seismic conditions, the vertical piles are sufficiently flexible to move with the silt should large soil movements occur. The vertical pile arrangement resisting all vertical forces and column moments was considerably less expensive than that using battered piles. The top of pile to footing connection is designed as a hinge to further increase lateral pile flexibility under seismic conditions.

7. AWARD OF CONTRACT

The contract for the construction of Special Structure S232 Outbound was awarded to Farmer Construction Ltd. of British Columbia in September 1983. The contract included work on an additional segment of the elevated guideway. The bid price for the special guideway structure was estimated at $350,000. Notice to proceed was given on 4 October 1983 and the contract completion date was set at 1 July 1984.

8. CONSTRUCTION

8.1 Foundations

From the geotechnical report the soil conditions were determined generally to be 2 m of fill overlying a variable layer of organic material, a thin layer of sand, and sound glacial till. From previous work in the area the fill was known to contain large timber and concrete obstructions and the till was known to contain large boulders of up to 1 m^3 in size. The sequence of construction for the piled foundations was excavation for the pile cap, installation of the precast shear key and backfilling, driving of piling, construction of pile cap, backfilling to top of pile cap, and lastly casting of the column. Observations from standpipes installed along the adjacent section indicated that the water table fluctuated between 2 m and 5 m below existing ground level. The installation of the 8.0 m (max) by 1.5 m deep by 0.5 m wide precast reinforced concrete shear keys, although below the water table, was achieved successfully by pumping, placing the shear keys and backfilling with granular backfill to the required 85% modified Proctor density.

8.2 Tendon Arrangement

To achieve maximum benefit from the post-tensioning, the tendons were designed for maximum eccentricities at mid-span and over the supports. Over the fixed support, 12 tendons from each end of the girder weave between each other through the support diaphragm reinforcement. The contractor was concerned that the tendon arrangement over the fixed support might cause congestion and thus hinder concrete placement and compaction. In response to the contractor's concerns, the section designer made a scale model of the box girder diaphragm over the fixed support to illustrate tendon clearance. This allayed the contractor's concerns and the model was kept in the engineer's office at the construction site for reference during tendon placement. To ensure efficient placing of concrete around post-tensioning ducts and reinforcement in the relatively narrow webs, super-plasticised concrete was used.

8.3 Grouting of Ducts

To remedy grout shrinkage voids at tendon high points, three vent hoses were installed at each high point and the shrinkage void was regrouted approximately 24 hours later. The second and third vent hoses were used for the second stage precautionary grouting. Steel rods were inserted in both tubes to avoid plugging during the first stage grouting.

8.4 Short-Term Shortening of Box Girder

The short term shortening of the concrete box girder due to post-tensioning, shrinkage, and creep was calculated to be 15 mm at column EC 227 and 35 mm at column EC 237. This shortening together with allowances for thermal expansion or contraction was compensated for during construction to obtain the required expansion joint gap setting.

8.5 Falsework

The subgrade supporting the falsework is progressively weaker from west to east along the line of the structure due to the increasing depth of the fill. To avoid conflicts with buried services and random obstructions in the fill the contractor elected to support the falsework on mudsills placed on compacted gravel and to monitor the structure for possible movements due to differential subgrade settlement. A jacking system was incorporated into the design of the falsework towers to permit vertical correction of the forms should settlement be experienced during the pouring of the box beam soffit slab. Upward adjustment, if required, was made prior to pouring the beam webs.

8.6 Formwork

The contractor elected to pour the box girder sections in four pours; bottom slab, webs, top slab and vertical parapets. The formwork was constructed of form ply and timber. As there was no designed access to the interior of the box sections the contractor was permitted to leave the forms for the top slab in the completed structure.

Conventional form construction was employed using cone ties, studs and walers. Curved surfaces were permitted to be constructed from straight segments with a maximum 6 mm offset from a true curve. To compensate for shortening of the structure under tensioning, removable timber strips were employed except for the top slab forms where compressible material was used. The tendons were tensioned in a predetermined sequence to minimize unbalanced effects. Falsework jacks were adjusted after stressing 50% of the tendons to compensate for movements under stressing and to avoid load concentrations developing on portions of the falsework.

8.7 Tolerances

To meet the stringent construction tolerances for the ALRT, the standard elevated guideway box beams were precast using an adjustable steel form under close shop supervision. For this cast-in-place box girder, a high level of survey accuracy was therefore required to maintain the same specified allowable variation in cross-section dimensions of -3 mm to +6 mm. The level of accuracy relative to the working survey control points was required to be:

- Any point on a tangent section to be established within ± 2 mm horizontally at 80% confidence level.
- On curved sections where control was from closest adjoining tangent, any point to be established within ± 5 mm horizontally at 80% confidence level.
- Elevation of any point to be within ± 2 mm of its theoretical elevation.

9. ACKNOWLEDGEMENTS

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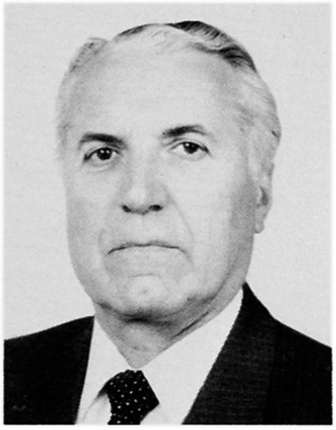
Les ouvrages du TGV

Die Kunstbauten des TGV

Structures of the TGV

Jacques GANDIL
Chef du Département
des Ouvrages d'Art
SNCF
Paris, France



Jacques Gandil, né en 1928, obtient son diplôme d'ingénieur civil à l'ENPC, Paris. Chef de la Division des ouvrages Lignes Nouvelles à la SNCF pendant 10 ans il a suivi tout particulièrement les études des ouvrages du TGV. Il dirige depuis cette année le Département des Ouvrages d'Art.

RESUME
La Société Nationale des Chemins de Fer Français vient de construire und ligne nouvelle à très grande vitesse entre Paris et Lyon, le TGV Sud-Est, en service depuis septembre 1981 resp. septembre 1983. Le présent article donne une description des ouvrages construits sur la ligne ainsi que diverses indications sur les conditions de leur surveillance et les constatations faites après plus de deux ans d'exploitation du tronçon Sud. Il donne enfin quelques informations rapides sur le nouveau projet du TGV Atlantique de la SNCF.

ZUSAMMENFASSUNG
Die Französischen Staatsbahnen (SNCF) haben eine neue Hochgeschwindigkeitslinie zwischen Paris und Lyon gebaut: die TGV-Linie Süd-Ost. Die Abschnitte Süd und Nord sind seit 1981 bzw. 1983 in Betrieb. Der Beitrag gibt eine kurze Beschreibung der Kunstbauten und Hinweise auf deren Unterhalt. Weiter werden die in den ersten Betriebsjahren gemachten Erfahrungen aufgezeigt. Schliesslich wird noch kurz das neue Projekt des „TGV-Atlantique" vorgestellt.

SUMMARY
The French National Railways (SNCF) have recently completed a new TGV-line (High-Speed Train) between Paris and Lyon. This article describes the structures of this line, as well as maintenance requirements based upon observations after two years of operation. The article also gives information on the design of the new TGV-Atlantic line.

1 – INTRODUCTION

Le présent article se propose de donner une description rapide des ouvrages du TGV Sud-Est, de donner quelques indications sur les conditions de leur surveillance et les constatations faites après plus de deux ans d'exploitation pour les ouvrages du tronçon Sud (275 km) mis en service en septembre 1981, de dire enfin quelques mots sur les ouvrages du T.G.V. Atlantique notamment sur les tunnels qu'il est prévu de construire dont certains seront parcourus à très grande vitesse.

2 – LES OUVRAGES DU TGV SUD-EST

2.1. Généralités

Le tableau 1 ci-après donne quelques indications générales sur les 500 ouvrages construits le long des 417 km de la ligne et de ses raccordements.

TYPE D'OUVRAGES			
Ponts-rails		Ponts-routes	
Viaducs	: 8		
Sauts de mouton	: 7		
Ouvrages spéciaux	: 30	Ouvrages spéciaux	: 15
Dalles en béton armé	: 20	Ponts-dalles en béton armé à trois travées	: 160
Tabliers à poutrelles enrobées	: 25	Autres ouvrages courants	: 10
Portiques et cadres en béton armé	: 100		
Petits ouvrages en béton armé	: 120		
Buses métalliques	: 25		
Dalots	: 20		
Total ponts-rails	: 355	Total ponts-routes	: 185
QUANTITES DE MATERIAUX EMPLOYES			
Béton armé	: 150 000 m³		
Béton précontraint	: 18 000 m³	Acier par béton armé	: 15 000 t
Coffrages	: 250 000 m²	Force de précontrainte	: 200 000 tf
Poutrelles métalliques	: 3 000 t	Drains poreux	: 40 000 m²

Tableau 1 : Types d'ouvrages du TGV Sud-Est. Quantités de matériaux employés.

On peut constater que près de 90 % d'entre eux sont de type courant alors que les ouvrages spéciaux représentent en nombre à peine 10 % du total.

La figure 1 donne quelques coupes transversales types.

Du fait de l'adoption de caractéristiques géométriques spécifiques (notamment rampes maximales de 35 %) et de l'amélioration considérable des techniques en matière d'exécution de remblais de grande hauteur et de tranchées profondes, le nombre des grands ouvrages est relativement limité : 8 viaducs seulement.

2.2. Particularités de conception

D'une manière générale, la conception des ouvrages a été orientée de façon à être assurée d'une excellente fiabilité des installations.

L'exploitation intensive de la ligne s'accommoderait mal en effet de fréquentes interventions sur les ouvrages d'art. Aussi, les types d'ouvrages ont été choisis avec le double souci de réduire l'entretien et de limiter les conséquences d'une défaillance locale.

Ainsi les ouvrages types « ponts-routes » sont des ponts dalles à trois travées en béton armé, qui, tout en ménageant une bonne « transparence » pour les mécaniciens offrent une grande réserve de sécurité vis-à-vis des efforts appliqués.

Les ponts-rails courants sont généralement des cadres ou des portiques monolithiques ou des ponts-dalles massifs à une travée.

Les ouvrages de plus grande portée comportent soit des tabliers à structures mixtes rustiques (poutrelles enrobées) soit des tabliers en béton précontraint.

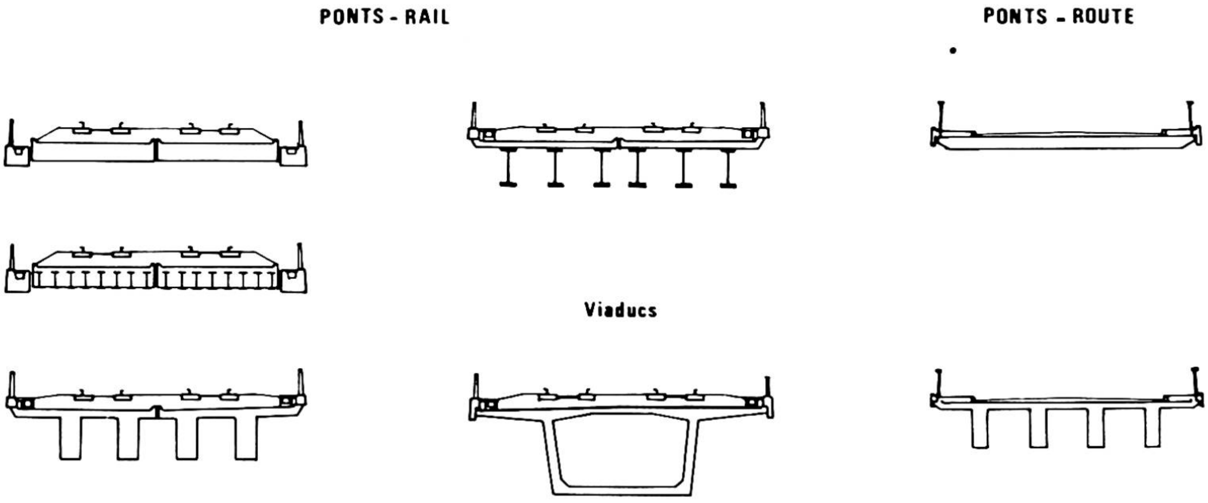


Fig. 1 Lignes TGV : coupes-types des ouvrages d'art

Certaines particularités de conception résultent directement d'autre part des effets des très grandes vitesses pratiquées (effets dynamiques, effets de souffle et effets sonores), on peut citer ainsi :

- le respect de gabarits spéciaux,
- le recours à des tabliers à déformabilité réduite, pour maîtriser notamment les phénomènes de résonance à certaines vitesses,
- la protection de la voie ferrée aux croisements fer-route : mise en place sur les ponts-routes de dispositifs prévenant la chute accidentelle de véhicules routiers et dans certains cas de dispositifs détectant les chutes de véhicules ou de leur chargement (fils détecteurs), adoption de diverses mesures pour les ponts-rails permettant de prévenir ou de limiter les conséquences de heurts de tablier par des véhicules de grande hauteur (voir figure 2).
- le traitement spécial des terrassements contigus aux culées à la transition « remblai-ouvrage » en vue d'en accélérer la stabilisation.
- enfin, le recours à des tabliers ballastés lourds (bien que la S.N.C.F. ait développé divers systèmes de pose de voie directe sur dalle) car le ballast est le matériau ayant les meilleurs qualités d'amortissement pour assurer un bon comportement dynamique de la voie aux très grandes vitesses prévues et pour les charges d'essieux des rames TGV (16,5 t) ; c'est aussi celui qui assure toutes choses égales par ailleurs, le niveau sonore le moins élevé. Une pose de voie sans ballast aurait exigé par ailleurs de multiplier les appareils de dilatation sur les voies en longs rails soudés.

2.3. Dispositions particulières propres aux ouvrages en béton précontraint. Cas des viaducs

Les viaducs ont fait l'objet de recherches de standardisation qui ont abouti, compte tenu de l'absence de trafic ferroviaire à la construction, à des structures précontraintes.

Les tabliers, à travées continues, sont généralement constitués par un caisson à deux âmes, dont l'épaisseur est de l'ordre de 1/14 de la portée et comporte des entretoises sur appuis. Les portées courantes varient de 44 à 50 m.

Les ratios sont, pour les tabliers, de l'ordre de 8 m^3 de béton au mètre linéaire, avec 100 à 120 kg d'armatures passives et 50 kg de precontrainte par mètre cube de béton.

A l'exception des ouvrages de faible hauteur (viaduc de franchissement de la Saône par exemple) les piles prévues sont des piles ayant une section transversale en H. La souplesse de ces piles a permis de réduire le nombre d'appareils d'appui mobile.

Les appuis d'extrémités peuvent être des palées enterrées dans le remblai d'accès à l'ouvrage ou des culées massives en béton.

Les efforts horizontaux, qui atteignent une valeur élevée (le seul freinage représente environ 1/7 de la surcharge roulante), sont ramenés sur un appui fixe particulièrement rigide, en général une culée, par l'intermédiaire d'un appareil d'appui fixe métallique.

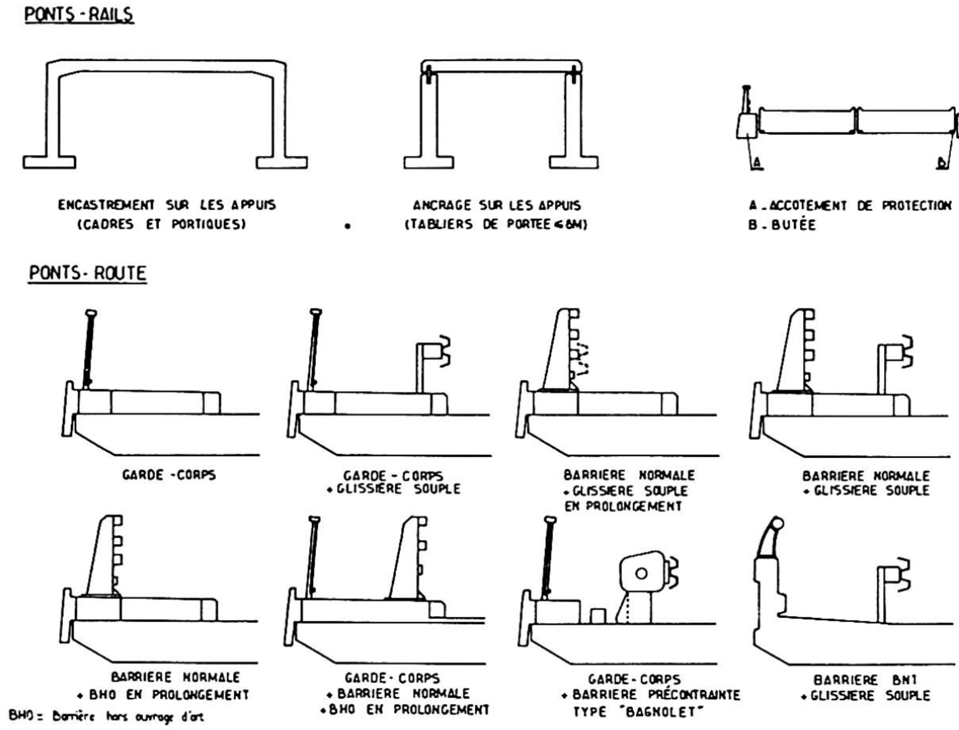


Fig. 2 Dispositifs de protection de la voie ferrée aux croissements fer-route.

Pour les viaducs de plus de 100 m de longueur dilatable, un appareil de dilatation de voie est prévu à l'extrémité côté appui mobile.

La conception des ouvrages tient compte de l'expérience des ouvrages routiers et satisfait aux prescriptions réglementaires en vigueur à l'époque de la construction. Elle satisfait en outre à un certain nombre de prescriptions complémentaires qui ont principalement pour but d'améliorer la fiabilité et la durabilité des ouvrages et qui concernent :

- les incertitudes sur le comportement des ouvrages,
- les points faibles de construction (reprises de bétonnage),
- la corrosion ultérieure des armatures,
- le remplacement des appareils d'appui.

Par exemple, dans les zones où les variations relatives de moments risquent d'être plus importantes que celles calculées, des câbles sont disposés aux fibres extérieures pour améliorer la résistance limite des sections.

Les réactions d'appui (au moins celles sur appuis extrêmes) ont été pesées et réglées à l'achèvement de la structure porteuse.

Dans les ouvrages coulés en place, les sections d'armatures sont renforcées au droit des reprises.

La tension maximale des armatures de précontrainte a été limitée entrainant ainsi une certaine diminution des risques de corrosion sous tension et corrélativement une augmentation des sections d'armatures et de la résistance limite des caissons (tension initiale $\leqslant$ 0,8 Tg ou 0,7 Rg, en général).

Les armatures sont dimensionnées pour qu'une perte de section de 30 % n'entraine pas la rupture sous surcharge majorée pondérée par le coefficient 1.3.

Par ailleurs, des bossages établis à la construction permettent la mise en œuvre d'une précontrainte complémentaire ultérieure telle que la structure satisfasse encore aux prescriptions réglementaires dans l'hypothèse où chaque armature de précontrainte d'origine aurait perdu 15 % de sa section.

Enfin, les appuis sont aménagés pour permettre le vérinage des tabliers et le remplacement des appareils d'appui.

La plupart des viaducs importants ont été construits par le procédé du poussage.

Ce procédé, en localisant les opérations délicates de construction du tablier sur une aire pouvant être aménagée confortablement à l'arrière d'une culée, est favorable à l'obtention d'une bonne qualité d'exécution des tabliers.

Avec les précautions énumérées plus haut, il concourt à doter les ouvrages de la fiabilité exigée par l'importance et la qualité du trafic que doit supporter la ligne nouvelle.

3 – LA SURVEILLANCE DES OUVRAGES

Outre les mesures classiques de surveillance (visites annuelles, inspections détaillées, visites spéciales éventuelles) les ouvrages de la ligne font l'objet de mesures spécifiques de surveillance. Ces ouvrages sont en effet soumis à des charges d'exploitation de caractéristiques particulières, plus agressives, pour certains ouvrages, de par la vitesse pratiquée (270 km/h) et la distribution répétitive des charges d'essieux (voir fig. 3) que celles sur lignes classiques.

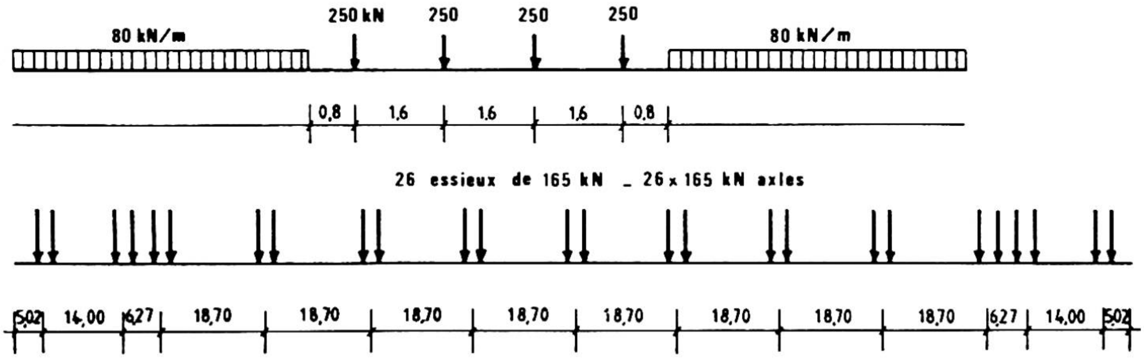


Fig. 3 Les schémas de charge UIC et TGV SUD EST

Des mesures spécifiques de surveillance ont donc été prises : surveillance spéciale d'analyse de comportement d'ouvrages courants, instrumentations particulières des grands ouvrages.

3.1. Surveillance spéciale pour analyse de comportement d'ouvrages courants

Certains tabliers isostatiques, classés par familles, ont fait l'objet d'une surveillance particulière : ponts à poutrelles enrobées, dalle en béton armé, tabliers bi-poutres en béton armé, de façon à suivre leur comportement sous charge d'exploitation et essayer d'apprécier leur plus ou moins grande sensibilité au vieillissement.

Les tabliers qui, dans chaque famille, apparaissaient comme les plus sensibles du fait de leurs caractéristiques (portées, fréquence propre, moindre amortissement) à l'agression des charges d'exploitation, ont ainsi été retenus, soit une vingtaine en tout.

Ces tabliers ont fait l'objet d'une inspection détaillée après deux ans d'exploitation et de diverses campagnes de mesures : flèches statiques et dynamiques, relevés d'accélération aux passages des circulations à vitesses différentes avec diverses compositions de rames : rame simple, rames doubles (voir figure 4).

3.2. Instrumentation particulière des grands ouvrages

Les dispositions classiques prévues pour assurer la surveillance ont été renforcées par un certain nombre de dispositions particulières :

– les massifs d'appui des piles et culées sont équipés de quatre repères permettant grâce à des nivellements de haute précision d'apprécier tout mouvement de fondations.

– les caissons sont équipés, dans toutes les travées, de jauges à cordes vibrantes (témoins COYNE), à mi-travée et sur appui, de façon à déceler toutes anomalies éventuelles dans la répartition des efforts;

– des repères de nivellement sont installés en divers points des tabliers de façon à apprécier leurs déformations sous charges statiques.

Il est prévu d'effectuer des inspections détaillées plus nombreuses en début d'exploitation que sur lignes classiques : une inspection chaque année pendant les deux premières années puis cycle d'inspection détendu progressivement par la suite pour en arriver au cycle normal de cinq ans.

Les principaux contrôles effectués sont : les examens visuels avec relevés, identification et marquage des fissures éventuelles, les mesures de contraintes (relevés bi-annuel des témoins COYNE), les nivellements des massifs d'appui et des tabliers, l'examen visuel et relevé des déformations des appareils d'appuis et joints de dilatation, l'examen du comportement de la voie et des divers éléments d'équipements au passage des circulations.

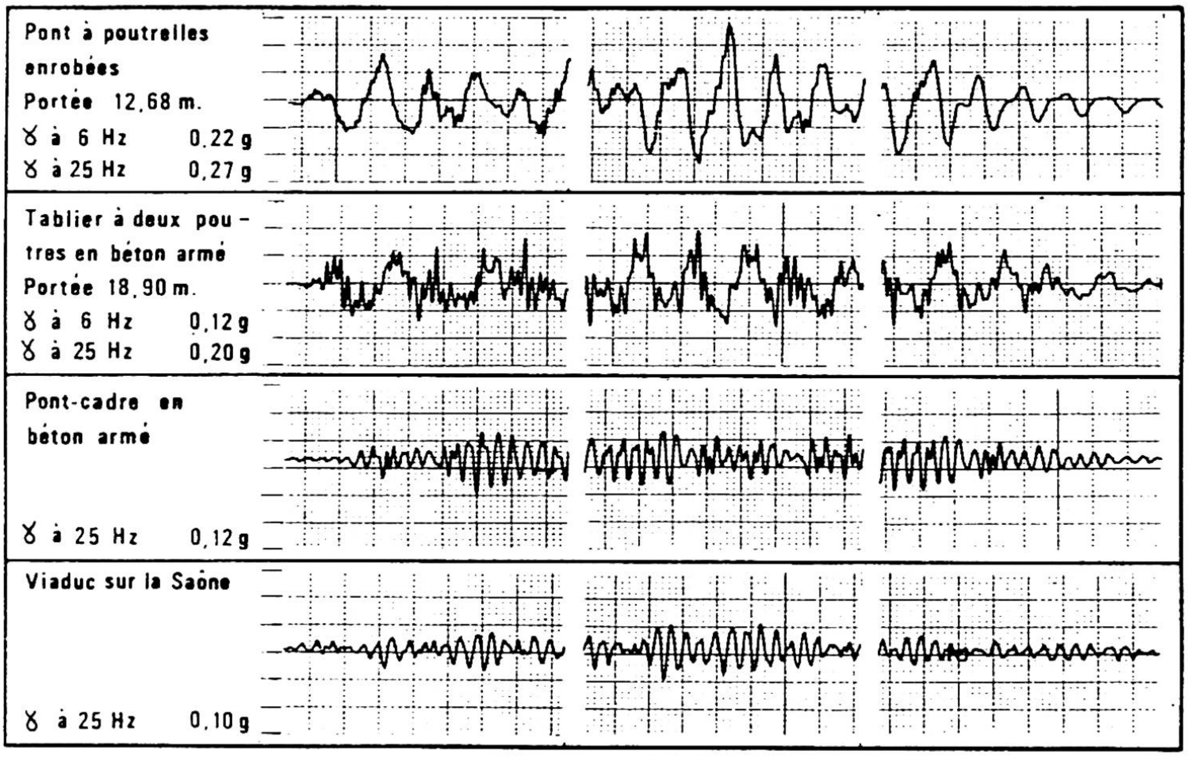


Fig. 4 Mesures d'accélération sur divers types d'ouvrages

Les résultats de mesures des témoins COYNE sont stockés en mémoires avec l'ensemble des mesures effectuées lors des relevés précédents puis traités informatiquement de façon, après avoir éliminé les phénomènes parasites liés aux fluages, retrait, relaxation, effets de la température et des gradients, etc..., à faire apparaître l'évolution des contraintes sous « charges permanentes + effet de précontrainte ».

Outre les diverses mesures exposées ci-avant, des mesures d'accélération ont été faites en certains points des tabliers pour apprécier les comportements vibratoires et leurs évolutions comme pour les petits ouvrages (voir figure 4).

4 – BILAN DES INSPECTIONS ET MESURES

Les diverses visites, inspections et mesures effectuées ont démontré que les ouvrages se comportaient de façon satisfaisante au passage des circulations, que les régimes vibratoires qui s'installaient étaient tout à fait compatibles avec une bonne tenue de la voie portée et a priori avec une bonne durabilité des structures.

Les ouvrages qui s'avèrent les plus sensibles au passage des circulations sont les ponts à poutrelles enrobées isostatiques, ce qui est très logique, du fait en général de leur moindre fréquence propre et de leur très faible coefficient d'amortissement.

Les campagnes de mesures systématiques visées en 3.1. ci-dessus ont montré une certaine évolution des ces tabliers, tout au moins pour ceux qui étaient les moins rigides, avec perte d'inertie, réduction concomitante de fréquence propre et par suite un état vibratoire moins satisfaisant au passage des circulations.

Une telle perte d'inertie n'a rien de surprenant. Sous l'action des circulations des effets de fatigue, des dégradations de natures diverses, le réseau de fissurations qui s'établit dans la partie tendue du béton devient plus dense, l'adhérence entre béton et poutrelles devient moins ténue, il en résulte une perte d'inertie. Ceci n'est pas nouveau et existe pour les ponts sur lignes classiques. On pourrait penser toutefois que, pour les lignes TGV où l'état vibratoire induit dans les tabliers par les circulations est nettement plus accentué que sur les lignes classiques, l'évolution de ce type de tablier soit plus précoce et plus profonde. C'est pourquoi un suivi particulier de quelques ouvrages de ce type est fait à l'aide de mesures de flèches statiques sous trains d'épreuve et de mesures d'accélération au passage des circulations.

Les mesures effectuées à ce jour montrent des évolutions variables selon les tabliers contrôlés avec apparition de stabilisation après des pertes d'inertie de 20 % en moyenne.

Pour les grands ouvrages, les mesures d'accélération et de contraintes effectuées font apparaître un comportement tout à fait normal. Une fois neutralisés les effets de fluage ainsi que ceux de la température, les contraintes sous « poids propre + précontrainte » n'évoluent pratiquement pas. La figure 5 donne un exemple d'évolution au droit d'une section de caisson.

Quatre éléments supplémentaires sont à ajouter au constat ci-dessus :

- aucun tassement différentiel significatif n'a altéré la qualité du nivellement de la voie aux abords des ouvrages, le traitement spécial des terrassements contigus aux culées évoqué en 2.2. ci-avant ayant correctement joué son rôle,
- les appareils de dilatation de la voie sur les viaducs ont donné lieu dans les premiers temps à des reprises de nivellements supplémentaires. Une amélioration de leur conception et l'utilisation d'un métal en nuance dure ont apporté une solution satisfaisante;
- malgré toutes les dispositions prises du point de vue tracé, visibilité, hauteur libre dégagée, un certain nombre de tabliers ont été heurtés par des véhicules routiers hors gabarit réglementaire. Grâce aux accotements de protection et au blocage des tabliers par butées aucun déplacement n'a été constaté.
- aucune chute de véhicules ou de leur chargement n'a eu lieu depuis un pont-route malgré quelques heurts des barrières de sécurité.

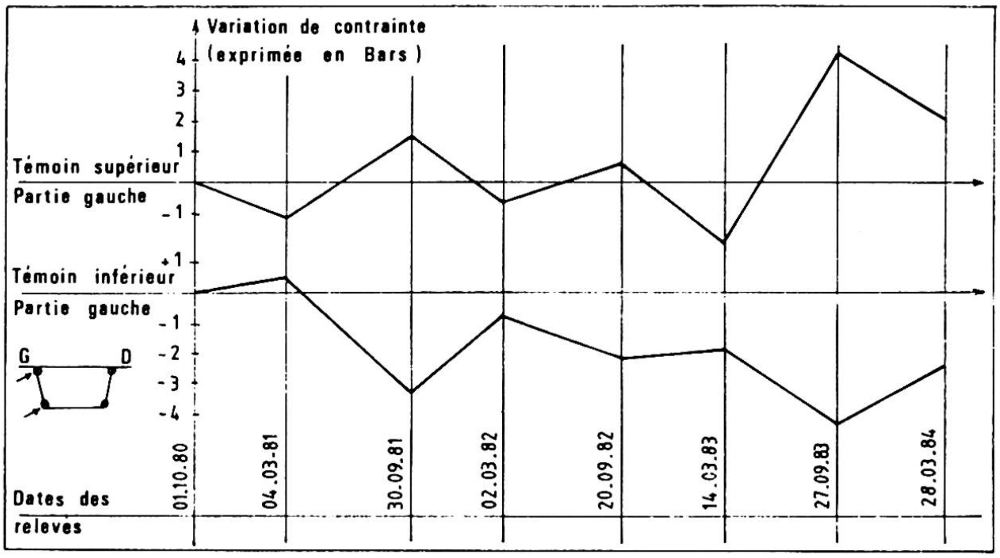


Fig. 5 Viaduc de la Digoine : variations des contraintes en milieu d'une travée

5 – UN NOUVEAU PROJET DE LA S.N.C.F. : LE TGV ATLANTIQUE

Devant la réussite commerciale que représente le TGV Sud-Est, le Gouvernement Français a autorisé la S.N.C.F. à entreprendre les travaux d'un nouveau projet intéressant 290 km de ligne destiné à relier la Région Parisienne et l'ensemble de la façade Atlantique de la Bretagne aux Pyrénées.

Pour ce qui concerne les ouvrages d'art, étant donné le bilan très positif exposé ci-avant, les conceptions et les méthodes d'exécution retenues pour le TGV Sud-Est seront reconduites pour l'essentiel.

Afin de réduire tout risque de phénomènes vibratoires un plus grand recours sera fait cependant aux ouvrages hyperstatiques ou aux tabliers à fréquence propre élevée.

Il convient de noter cependant que, en raison de l'adoption de caractéristiques géométriques réduites (rampes maximales de 15 ‰ – exceptionnellement 25 ‰) et la nécessité de « s'enterrer » en zones urbanisées, dans la proche banlieue parisienne en particulier, la construction d'importants ouvrages souterrains est prévue :

- 4 tunnels d'une longueur cumulée de 7 200 m, dont un de 4 800 m de long, comportant deux tubes à 1 voie,
- 5 tranchées enterrées d'une longueur cumulée de 6 100 m.

En outre 15 viaducs ou estacades, totalisant environ 5 100 m sont prévus soit une quantité plus de deux fois supérieure à celle rencontrée sur le TGV Sud-Est.

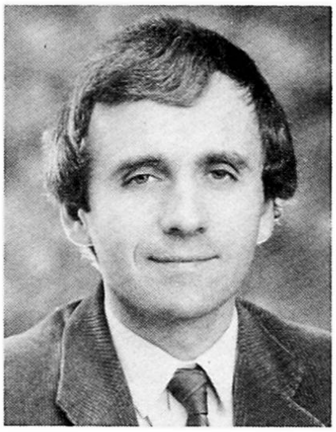
Certains des souterrains devant être parcourus à grande vitesse, la S.N.C.F. a développé tout un programme d'études et d'essais afin de maîtriser le mieux possible les problèmes divers liés à la pénétration et au franchissement de souterrains de grandes longueurs à très grandes vitesses.

Viaducs et stations aériennes du Métro de Marseille

Viadukte und überirdische Stationen der U-Bahn in Marseille

Aerial Structures of the Marseilles Urban Rapid Transit System

Michel CROC
Directeur Technique
Société du Métro
Marseille, France



Michel Croc, né en 1948, est ancien élève de l'Ecole Polytechnique de Paris et Ingénieur des Ponts et Chaussées. Après avoir été maître d'oeuvre de projets portuaires, routiers et aéroportuaires, il est actuellement responsable technique de la construction des lignes de métro de Marseille.

RESUME

L'infrastructure du Métro de Marseille a été construite en souterrain profond, en tranchée couverte, au sol et en viaduc. Ces quatre modes de construction ont chacun des avantages propres qui les recommandent l'un ou l'autre en fonction des conditions locales. Ce texte présente les bases de l'analyse, l'illustre par les choix effectués à Marseille, et décrit plus particulièrement les viaducs et les stations aériennes du réseau de cette ville.

ZUSAMMENFASSUNG

Die Linie der U-Bahn in Marseille führt durch Tunnel, durch überdeckte Einschnitte, über offenes Gelände und über Viadukte. Jede dieser Konstruktionsarten hat ihre spezifischen Vorteile, die entsprechend den örtlichen Gegebenheiten für ihre Wahl entscheidend sind. Dieser Artikel stellt die Grundlagen der Analyse vor, erklärt sie durch die hier jeweils gewählten Lösungen und beschreibt im besonderen die Viadukte und überirdischen Stationen des U-Bahnnetzes.

SUMMARY

Marseilles "metro" infrastructure has been built deep underground, in cut and cover, at grade and on aerial structures. These four modes of construction each have their own advantages that allows for selecting one or the other, depending on local conditions. This paper gives the basis for analysis, presents the choices that have been made in Marseilles and describes, in particular, the aerial structures and stations existing in Marseilles network.

1 – PRESENTATION DU SYSTEME DE TRANSPORTS COLLECTIFS EN SITE PROPRE DE MARSEILLE

1.1. Le réseau

Marseille a fêté ses vingt cinq siècles d'histoire. Le réseau de transports de surface s'adapte difficilement à l'ancienne structure urbaine du centre, et l'extension de l'habitat en périphérie n'a fait qu'aggraver le problème en allongeant les distances.

C'est pourquoi la ville a décidé en 1964 de créer un réseau de transports collectifs en site propre qui réponde à trois objectifs :

– desservir le centre ville en assurant des liaisons directes avec l'habitat et l'emploi,

– bien assurer les correspondances avec les autres modes de transport,

– être extensible au fur et à mesure des possibilités et des besoins.

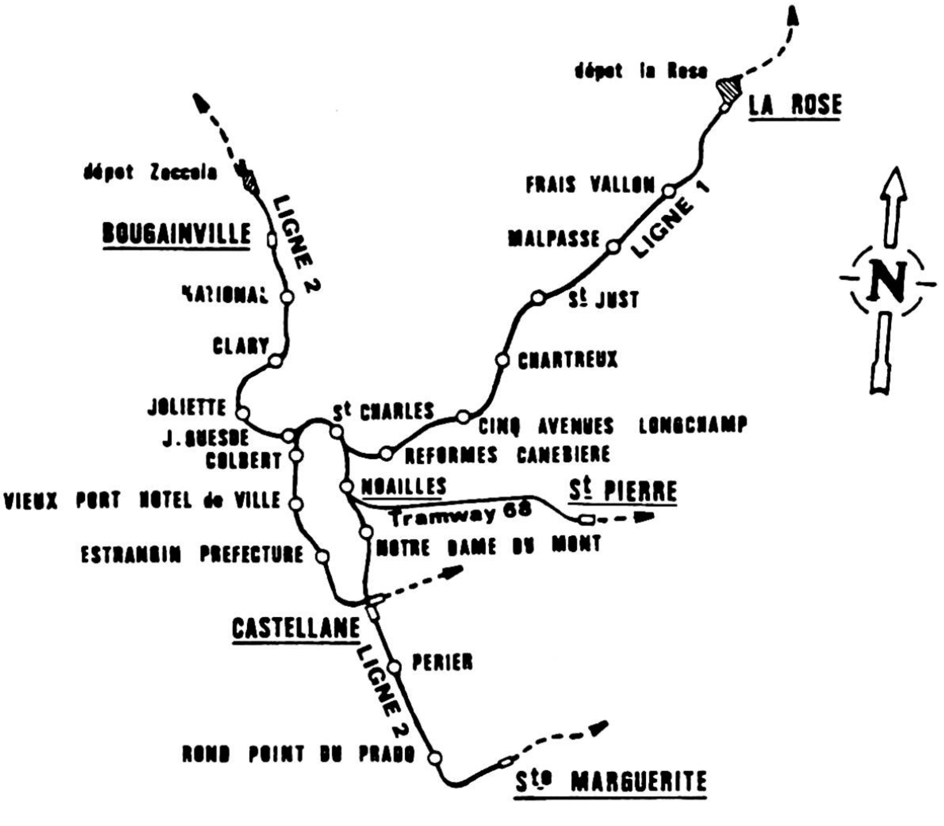


Fig. 1 Le réseau et ses extensions

Les premiers éléments consistent en 2 lignes qui ceinturent le centre et lancent quatre antennes vers les pôles périphériques. Ces deux lignes actuellement en service (La Rose – Castellane et Joliette – Castellane) ou en construction (Joliette – Bougainville et Castellane – Sainte-Marguerite) seront prolongées pour donner l'accessibilité par les transports collectifs à des quatiers où le bénéfice en sera considérable.

La première ligne (La Rose – Castellane), longue de 9 km, comporte douze stations. Sur les 3 km de la partie aérienne, on trouve deux sections en viaducs, longues de 690 m et 120 m.

La deuxième ligne (Bougainville – Sainte-Marguerite), longue de 9 km, comporte douze stations. On y rencontre 670 m de viaducs au Nord et 620 m au Sud.

1.2. Le matériel roulant et la voie

Les trains du métro de Marseille se composent de trois (puis quatre) voitures. La capacité normale (4 personnes debout par mètre carré) de ces trains est de 352 (resp. 472) voyageurs dont 136 (resp. 184) assis.

Les trains roulent sur pneumatiques d'axe horizontal, et sont guidés par des pneumatiques d'axe vertical. Les bogies comportent des roues fer qui servent de tambours de freins, dont les mentonnets assurent le guidage dans les appareils de voie, et qui servent de roues porteuses en cas de dégonflement des pneus ou dans certaines zones d'atelier.

La voie, posée sur béton en souterrain et sur ballast en aérien (y compris les viaducs), se compose de traverses (mixtes béton acier sur chaussons élastomères en souterrain, bois en aérien), de deux rails fer (50 kg/ml), de deux pistes de roulement des pneumatiques (profilés en I de 68 kg/ml), et de deux barres latérales de guidage (cornières métalliques de 44 kg/ml).

2 – COMPARAISON ENTRE LE PASSAGE EN GALERIE, EN TRANCHEE COUVERTE, EN VIADUC, AU SOL

Nous allons étudier les critères de comparaison et de choix entre les quatre modes de construction possible, en les illustrant par des applications à Marseille.

2.1. Coûts unitaires

On peut comparer les coûts dans des conditions moyennes de difficulté, en prenant bien garde de ne pas limiter la comparaison aux prix de génie civil, mais d'y ajouter ceux des acquisitions foncières, des déplacements de réseaux, des traitements de terrains et des équipements, tout en excluant ceux des ouvrages non linéaires (poste de commande centralisée, stations, ateliers, matériel roulant). Cela donne (MMF 1984 au km de ligne) :

	génie civil	total
au sol	30	90
en viaduc	50	110
tranchée couverte	80	200
galerie	100	190

On constate que dans ce cas moyen, les prix de la voie au sol et en viaduc d'une part, en tranchée couverte et en galerie d'autre part, sont très proches.

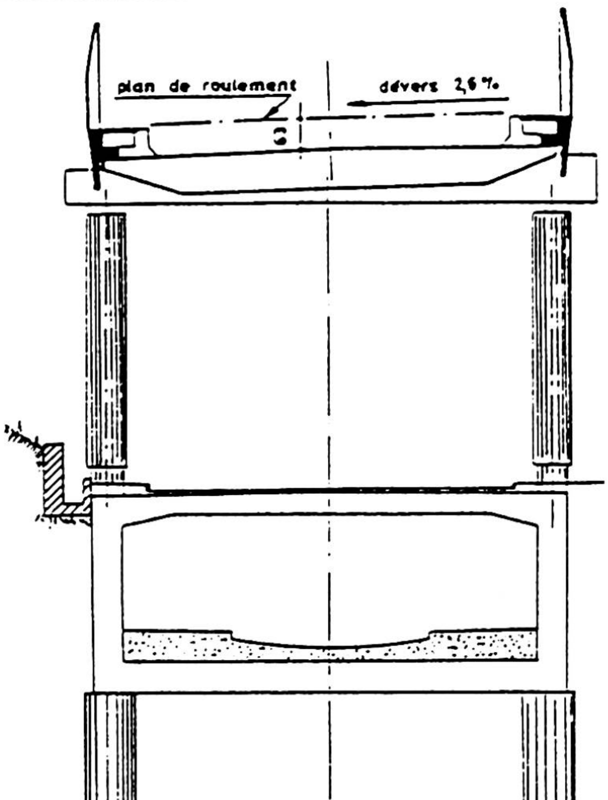


Fig. 2 Canalisation, route et métro superposés

2.2. Géologie

Il est clair que l'on peut classer dans l'ordre de sensibilité croissante aux difficultés géologiques : au sol, en viaduc, en tranchée couverte, en galerie.

2.3. Hydrologie

La présence d'un cours d'eau, traversé au suivi par un projet de ligne, est un facteur important. Par exemple suivre le lit d'un cours d'eau en tranchée couverte n'est pas possible, le suivre en viaduc peut permettre de profiter d'une trouée dans l'urbanisation.

Si la mauvaise qualité des eaux est telle qu'il vaut mieux couvrir le cours d'eau, une solution qui a été adoptée à Marseille sur le tronçon Nord de la deuxième ligne consiste à construire un ouvrage qui superpose la canalisation du cours d'eau, une voie routière, et le métro en viaduc.

2.4. Densité du bâti

Si la ville ne possède pas d'avenue de grande largeur on est obligé de construire des galeries. Ainsi dans tout le centre de la ville de Marseille, l'étroitesse des rues, sinueuses et sans alignement les unes par rapport aux autres, et le caractère très accidenté du relief, n'ont pas permis de trouver un tracé à fleur de sol comme à Paris ou à Lyon. Il a donc fallu s'enfoncer profondément afin de passer en tréfond des immeubles, et creuser des galeries à une seule voie. Lorsque les conditions étaient un peu moins sévères, on a creusé des galeries à deux voies.

2.5. Environnement

L'impact sur l'environnement doit s'évaluer pendant l'exécution des travaux, et après. Un projet au sol peut s'admettre dans le cadre d'un projet routier concomitant, un projet en viaduc ne pose question que dans des zones de grande qualité naturelle ou architecturale.

2.6. Projet routier concomitant

On peut développer une conception d'ensemble très intéressante et économique si l'on programme simultanément la réalisation d'un projet routier et d'un projet de métro. On en trouve trois exemples caractéristiques à Marseille :

- deux stations (Malpassé et Frais Vallon) et presque deux interstations sur le terre-plein central d'une autoroute radiale, au sol,
- une station (National) est construite en tranchée couverte dans une trouée qui permettra l'ouverture d'une rue nouvelle,
- une interstation (Bougainville – Dépôt Zoccola) superpose la canalisation d'un ruisseau, une rue nouvelle, et le viaduc du métro.

2.7. Positionnement des ateliers

Les ateliers et garages doivent être aériens pour un coût raisonnable. La recherche d'une implantation judicieuse, limitant les raccordements techniques parcourus sans voyageur, conduit souvent à des sections terminales au sol ou en viaduc.

3 – PRESENTATION DES VIADUCS DU METRO DE MARSEILLE

3.1. Une demi-interstation et un terminus en service depuis 1977 (Frais Vallon – La Rose)

La station terminus de La Rose (116 m y compris les culées) est composée d'une travée indépendante de 22 m, et d'un viaduc à trois travées continues de 79 m.

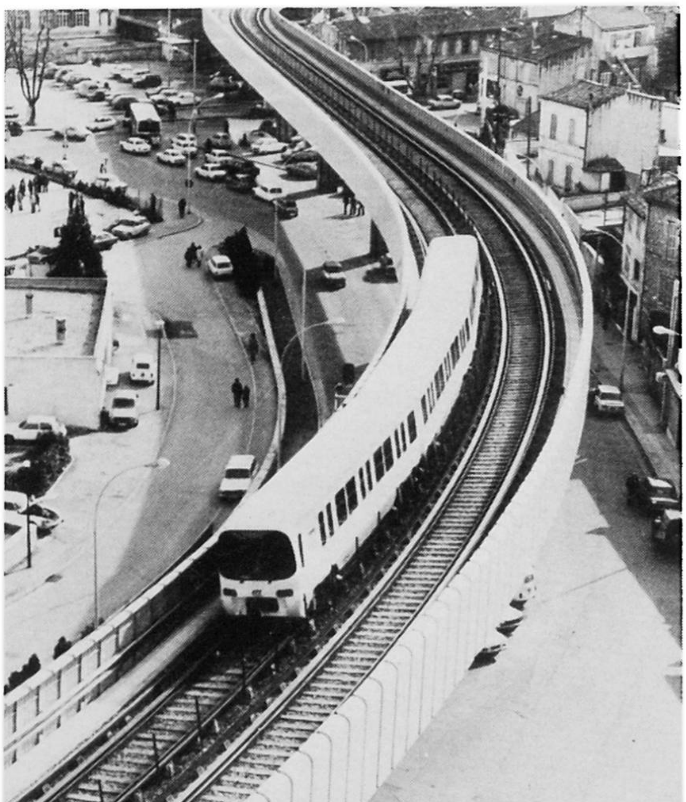


Fig. 3 Le viaduc de La Rose

L'ossature porteuse du tablier est constituée par un double caisson en béton précontraint avec encorbellements.

Le viaduc courant (564 m et une culée de 10 m) est constitué de 5 ouvrages successifs, soit quatre ouvrages à quatre travées de 27,75 m, et un ouvrage à quatre travées de 30 m. L'ossature porteuse du tablier est constituée par une poutre caisson en béton précontraint comportant deux encorbellements symétriques. (Entreprises QUILLERY, MOINON, CAPAG-CETRA et HEULIN).

3.2. Une demi-interstation et un terminus en construction ; mise en service en 1986 (Rond-Point du Prado – Sainte-Marguerite)

La station terminus de Sainte-Marguerite (longue de 70 m) est composée de deux travées indépendantes de 30,25 m de portée chacune, reposant sur trois cantelivers longs chacun de 10,35 m. Les cantelivers sont des ouvrages massifs précontraints longitudinalement et traversalement. Les tabliers sont des doubles caissons en béton précontraints longitudinalement.

Les viaducs courants (longs de 550 m y compris les culées) de part et d'autre de la station sont au nombre de sept, dont six de trois ou quatre travées de longueur 23,40 m (une travée), 24 m (neuf travées) et 25,79 m (neuf travées) et un isostatique de 15,20 m (une travée). L'ossature porteuse du tablier est constituée d'une poutre caisson en béton précontraint comportant deux encorbellements symétriques (Entreprise SOBEA).

3.3. Une interstation et demie et un terminus en cours d'études d'exécution, mise en service 1987 (National – Bougainville – Dépôt Zoccola).

La station terminus de Bougainville est composée d'un tablier en dalle précontrainte à quatre travées de 17,50 m et d'un bâtiment de 20 m. Les tabliers sont précontraints longitudinalement.

Les estacades courantes (d'une longueur de 580 m plus deux culées de 10 m chacune) de part et d'autre de la station sont au nombre de six, composés de cinq ou six travées longues de 18 m (quatre viaducs) ou 18,25 m (deux viaducs) ; sur une longueur de 329 m, les estacades ont des fondations communes avec le cuvelage du ruisseau, lui-même couvert et porteur d'une chaussée routière. Le cuvelage est un cadre en béton armé, le tablier une dalle précontrainte longitudinalement (Entreprises BOUYGUES, BOUYGUES OFFSHORE et MISTRAL TRAVAUX).

4 – CARACTERES PARTICULIERS D'UN VIADUC METRO

4.1. Règlement de calcul et cas de charge

Les règlements français de calcul du béton armé et du béton précontraint sont des règlements aux états limites. Il faut cependant adapter des cas de charges aux spécificités des ouvrages du métro. Le convoi type est caractérisé par des efforts de freinage et d'accélération importants. Par ailleurs, un ouvrage comme celui représenté par la figure 2 est composite et soumis à des sollicitations de natures très différentes.

4.2. Mouvements de la voie

Les profilés métalliques qui constituent la voie forment un ensemble très hétérogène, soumis à des déformations et des efforts très divers. On n'a pas pu démontrer la faisabilité d'une pose directe de la voie pneu métro sur viaduc et l'on utilise le ballast, matériau élastique, déformable, et que l'on peut entretenir et régler.

4.3. Profil en travers

La voie métro est parcourue par les câbles qui transportent les courants forts et ceux qui transportent des courants faibles. La disposition adoptée à Marseille en viaduc est de poser les premiers dans des caniveaux de béton latéraux, et les seconds sur des tablettes métalliques fixées à l'intérieur des garde-corps antibruit. Le cheminement d'évacuation des passagers en cas de panne immobilisant un train en ligne passe alors sur les caniveaux de câbles courants forts, qui

Fig. 4 Le Viaduc de Sainte-Marguerite

doivent être couverts de dallettes stables. Les signaux optiques des zones de manœuvre sont fixés sur le garde-corps antibruit. Les appareils de voie doivent être placés sur des zones rectilignes, planes, leur plan étant légèrement incliné pour le bon écoulement des eaux.

4.4. Protection antibruit et esthétique

Pour la protection des riverains et de ceux qui cheminent sur les voies, des garde-corps antibruit sont installés de part et d'autre des voies. Comme ils enveloppent des boggies, zone des sources de bruit, ils sont très efficaces. La forme et la matière de ces garde-corps ont été définies par les architectes.

Mais l'intervention des architectes ne se limite pas à cela. Ils sont chargés, à partir des schémas fonctionnels, d'étudier des aménagements qui donnent aux zones d'accueil du public un caractère humain, accueillant et confortable et plus particulièrement de respecter la simplicité des volumes tout en compensant leur éventuelle rigueur par des recherches picturales et par l'animation.

Les viaducs ont été mis au point dans le même esprit de simplicité des formes et lignes, avec recherche dans les matériaux (par exemple piles cannelées, agrégats de béton rendus apparents par lavage ou décapage, garde-corps en béton blanc ...).

Quant aux stations terminales (Bougainville et Sainte-Marguerite), elles ont été conçues comme des volumes comptant très fort dans l'environnement, et la griffe de l'architecte y sera très visible.

4.5. Structure et ambiance climatique des stations

Une station de métro aérienne est un cas assez rare de viaduc où des personnes stationnent quelques minutes par jour, beaucoup plus longtemps aux heures creuses de la nuit. Plutôt que de résoudre le problème en refermant totalement la zone des quais, on a mis au point à Marseille un système d'auvents ajourés qui tempèrent les éléments sans les faire disparaître.

4.6. Traitement des terminus en viaduc

Si l'on veut éviter la dépense d'une longueur de viaduc au-delà du terminus qui assure la distance d'arrêt des trains en toute sécurité, il faut prévoir comme à Sainte-Marguerite un heurtoir qui offre au choc une résistance croissante, arrêtant le train sans dégât ou à la limite avec des dégâts réparables.

5 – CONCLUSION

A cause de son implantation dans les zones denses de ville dont le centre est souvent ancien, le métro est en général souterrain. Dans les zones périphériques en revanche, il peut être beaucoup plus économique et admissible de le construire en viaduc, grâce à la discrétion et parfois la beauté auxquelles peuvent atteindre des ouvrages aériens. Nous espérons l'avoir montré au travers de cette description du Métro de Marseille, élargie aux caractéristiques de tout métro aérien.

Fig. 5 Le viaduc et le terminus de La Rose

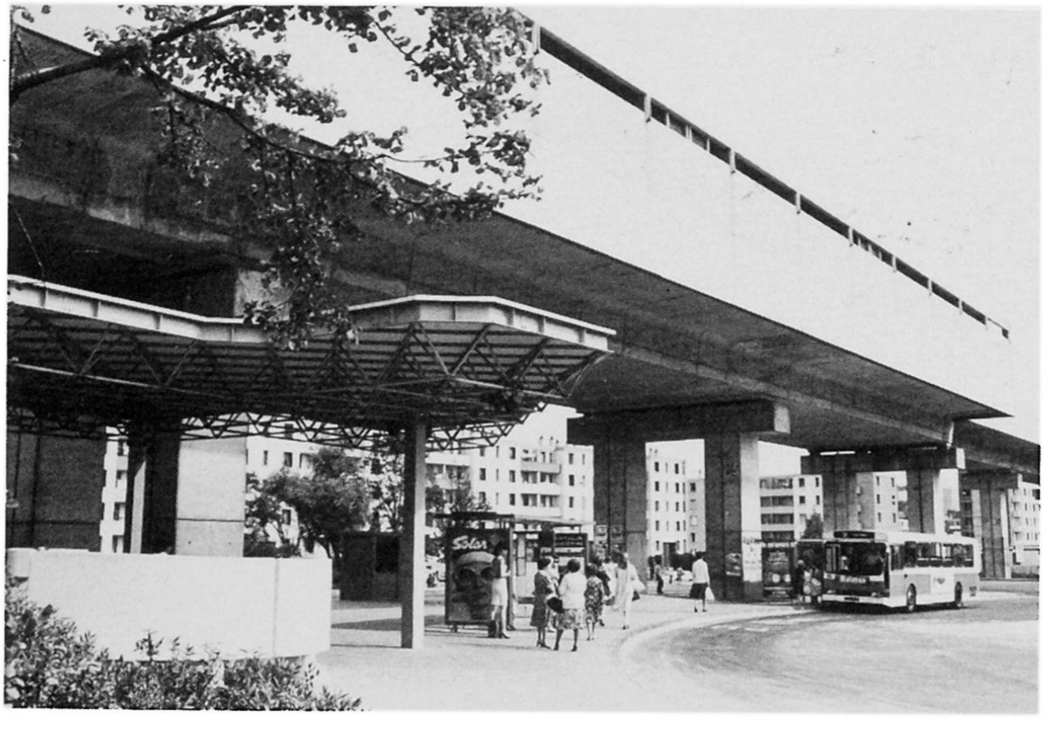


Fig. 6 Le terminus de La Rose

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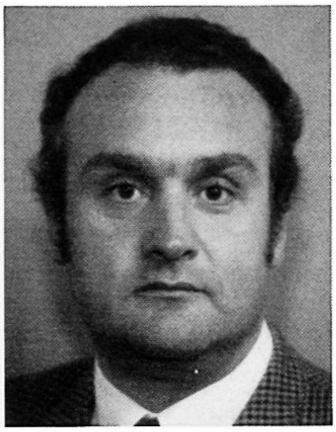
Transmission of Longitudinal Forces on Railroad Bridges

Transmission des forces longitudinales dans les ponts-rails

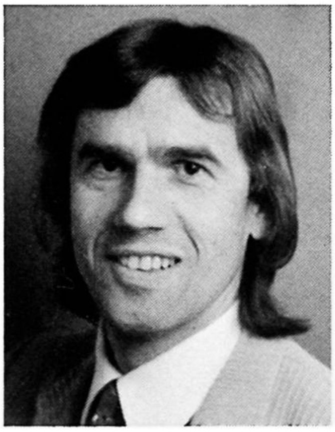
Tragsysteme zur Abtragung von Längskräften auf Eisenbahnbrücken

Gerhard PROMMERSBERGER
Civil Eng., Director
German Federal Railway
Karlsruhe, Fed. Rep. of Germany



G. Prommersberger, born in 1941, Dipl.-Ing. in Civil Engineering, from the Techn. Univ. of Darmstadt. Member of the Subway Dpt. in Frankfurt/Main. From 1973 to 1976 Scientific Officer in the German Federal Railways. Since 1976 Dir. of Bridges and Tunnels for the Project Group Mannheim-Stuttgart in Karlsruhe.

Richard ROJEK
Project Eng.
Leonhardt und Andrä, Consult. Eng.
Stuttgart, Fed. Rep. of Germany



Richard Rojek, born in 1944. Received his Dipl.-Ing. degree in Civil Engineering from the Univ. of Stuttgart in 1970. Since that time he has worked with Leonhardt und Andrä as a Project Engineer on all aspects of the design of bridges, soil-engineering, and design of buildings.

SUMMARY
The paper outlines which structural systems are possible and suitable for the transmission of longitudinal forces on railroad bridges. Their dependence on other design and construction parameters is shown. The different systems are defined by significant limiting forces and are evaluated.

RESUME
La contribution met l'accent sur les systèmes structuraux appropriés pour la transmission des forces longitudinales dans les ponts-rails. Ces systèmes dépendent aussi d'autres paramètres de projet et de construction. Ces divers systèmes sont décrits et discutés, notamment sur la base de valeurs significatives pour les charges.

ZUSAMMENFASSUNG
Mit dem vorliegenden Beitrag wird aufgezeigt, welche Tragsysteme für die Abtragung der Längskräfte auf Eisenbahnbrücken aus heutiger Sicht möglich und sinnvoll sind. Ihre Abhängigkeit von anderen Planungs- und Konstruktionsparametern wird aufgezeigt. Die verschiedenartigen Systeme werden unter anderem durch signifikante Lastgrössen abgegrenzt und bewertet.

1. INTRODUCTION

When planning for the new railroad lines of the German Federal Railway (Deutsche Bundesbahn, DB) started in 1975, the design for bridges with respect to vertical loads could be based on the state-of-the-art of road bridges although these loads are significantly larger. The design for longitudinal forces, however, could be based only to a very limited extent on the experiences from road bridges. The reasons are on one hand that the size of the braking and acceleration forces are considerably different, and on the other hand that the rails of railroad bridges are unintentionally cooperating load-bearing members for longitudinal forces or temperature restraints.

2. LONGITUDINAL FORCES ON RAILROAD BRIDGES

2.1 Braking and Acceleration Forces

New measurements have indicated that friction factors in excess of 40 % may be activitated shortly before a train stops. Consequently, the German Railway Codes require for the new railroad lines to take into account a braking force of 20 kN per m of track (friction factor 25 %) over a maximum length of 312,5 m. Onto these braking forces an acceleration force of 1000 kN has to be superimposed for double-track bridges over a loaded length not exceeding 30 m. For long bridges the biggest braking and accelerating forces are consequently

$$L_B + L_A = 6.25 + 1.0 = 7.25 \text{ MN}.$$

The corresponding longitudinal force for road bridges in Germany always remains smaller than 0.9 MN due to the individual traffic units.

2.2 Movement Resistance of Bearings

Changes in length of structures created by changes of temperature activate resistance in the bearings. In order to quantify the corresponding loads, a double track single cell prestressed concrete box on sliding bearings with a span of 44 m is used as a typical example: the rails with ballast and the structural concrete have a dead weight of about 190 + 260 = 450 kN/m. Assuming a friction factor of 3.5 % - the actual friction factors are given by the bearing manufacturers - each meter of superstructure supported on sliding bearings creates a friction force of 0.035 x 450 = 16 kN/m. For a 44 m span we reach

$$L_{F,S} = 0.035 \times 0.45 \times 44 = 0.7 \text{ MN}.$$

2.3 Restraint Forces from Continuously Welded Rails

As the structure transmits forces through the ballast to the rails, changes in length of the structure may cause restraint forces in the longitudinal direction. Depending on the bridge system the overall change of temperature in the structure or the temperature differential between structure and rails may be governing. The size of these restraint forces depends heavily on the structural system and will be discussed later. The changes in overall temperature given by the codes are -30 K and +20 K against the average temperature. As the temperature variations in the rails have to be accounted for with 50 K, the temperature differentials between structure and rails come to -20 K and +30 K.

2.4 Other Longitudinal Forces

Other longitudinal forces may be created by the fixed support of a superstructure on two or more piers, by unsymmetric sun radiation, rotations of foundations and wind loads on piers rigidly connected with the superstructure. These longitudinal forces are, however, small when compared to those mentioned earlier. Earthquake loadings are also neglected here due to their extremely low probability in Germany.

3. STRUCTURAL SYSTEMS WITH SINGLE SPAN GIRDERS

Experience indicates that for concrete bridge girders fixed at one end with spans up to about 90 m the continuity of the rails has not to be interrupted in order to cater for the changes in length of the superstructure. From this knowledge the structural concept was derived to design longer bridges as a sequence of single spans over which the ballast with the rails is carried continuously. This basic concept developed from the point of view that the railway tracks and the vertical load path offer various possibilities for carrying the longitudinal forces which are determined mainly by the topography (pier heights) and the stiffness of the foundations depending on the soils characteristics.

3.1 The Elevated Superstructure

The elevated superstructure should be understood with the idea that the transmission of longitudinal forces in simple span bridges have such a small influence on substructure deformations, that the structure can be regarded as nearly rigid. Under this condition the braking and acceleration forces acting on a span are completely transmitted to its fixed point pier. This load case does not cause any considerable stresses and strains in the rails. Each pier has to carry the same longitudinal force which for single and double track bridges with the often chosen spans of 25 m comes to

$$L_B + L_A = (20 + 33.3) \cdot 25 \cdot 10^{-3} = 1.3 \text{ MN}.$$

As the continuously welded rails cannot follow the changes in length of the beams they are restrained by temperature changes in addition to directly applied temperature loadings. The maximum values always occur at the beam joints and amount normally to

$$R_R = q\ell/4$$

with q meaning the resistance against sliding of the complete railtie grid or of the rails alone, see Fig. 1.

Special situations are created at the bridge ends: at the abutment with sliding bearings the restraint is about double its regular value, while at the abutment with the fixed beam connection the restraint decreases accordingly. On the abutment with the fixed point and on the first pier in front of the abutment with sliding bearings act - as a consequence - also longitudinal forces of the size R_R, which means for two rails and a span of 25 m

$$L_R \sim 2 \times 0.02 \times 25/4 = 0.25 \text{ MN}.$$

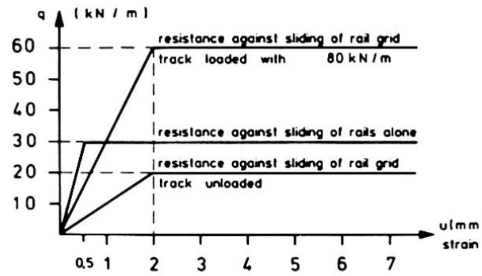


Fig. 1 Stress-strain diagram of ballast

The resistance forces from bearings have even smaller values for this system and may be neglected in accordance with the German Railway Code. Using the former calculation examples and taking into account the slightly smaller structural dead weight we reach

$$L_F \sim 0.035 \times 0.40 \times 25/2 = 0.18 \text{ MN}.$$

The loads to be applied to the structure are given in Fig. 2.

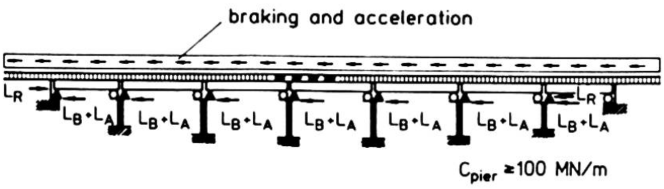


Fig. 2 Statical system and longitudinal forces of the elevated superstructure.

3.2 Valley Bridges without Special Devices

The formerly assumed simplification to neglect the pier deformations due to longitudinal forces is only valid for pier stiffnesses in excess of 100 NM/m which vary by not more than 10 % in between them. This condition can actually be fulfilled for pier heights up to about 15 m.

Higher piers and, more importantly, variations in stiffness between adjacent piers and abutments result in a statically highly redundant system. The continuously welded rails act unintendedly as a link between all the substructures of different stiffness and are, in addition, strongly anchored to the regular track continuations beyond both bridge ends.

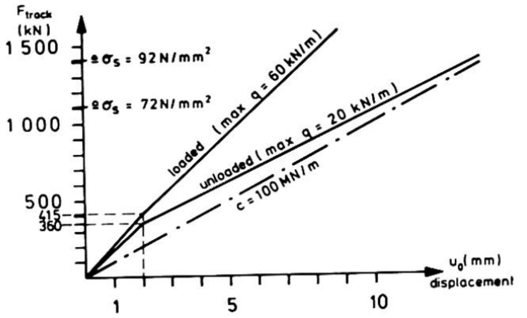


Fig. 3 Force-displacement characteristics of rail grids

The stiffness of this anchorage comes to 100 MN/m per rail grid for the range of permissible rail stresses as shown in Fig. 3. For pier heights up to 30 m the substructure stiffness is significantly influenced by the soils characteristics, which means that the structural assumptions for the proof of safety may suffer certain uncertainties.

Statical models become especially complicated and time-consuming because of the non-linear stress-strain characteristic of the ballast connecting the rails with the structure, see Fig. 1.

The load distribution by the tracks creates additional rail stresses which, if superimposed onto those due to change of temperature in the structure, must not exceed 72 MN/mm^2 in compression (safety against buckling of the rail grid) or 92 N/mm^2 in tension (safety against rail failure). At the same time it has to be demonstrated that the relative movements between rails and structure do not exceed 4 mm in order to ascertain a reliable support of the rails by the ballast.

The load-bearing system causes bigger loads at the stiffer abutments and piers than those loads actually acting on the corresponding spans.

The upper limit of the share of forces from braking and acceleration for very stiff abutments is obtained by adding to the load acting on the adjacent span that share of braking forces which the rails may carry across the next joint within the limit of permissible stresses. With a stress of about 70 N/mm^2 (excluding temperature influences), a rail area of about 2x77 cm^2 and 44(58) m spans, thus leads to

$$\max L_{B+A} = (1.0+0.02\text{x}44) + 70\text{x}154\text{x}10^{-4} \sim 3.0\ (3.24)\ \text{MN}.$$

The less stiff piers have to be loaded with at least one half of their direct load, i.e. for a 44 m span

$$\min L_{B+A} = 0.5\cdot(1.0 + 0.02 \times 44) = 0.94\ \text{MN}.$$

The pier stiffness influences only very little the restraint due to changes in temperature. With similar support conditions as those for the elevated superstructure, similar strains are to be expected here also.

For small pier heights (up to about 20 m) and favorable soil conditions it is appropriate to provide fixed-sliding support conditions similar to those of the elevated superstructure as shown in Fig. 4a. For this system the extreme rail stresses and track movements will occur at the joint at the abutments. If the permissible values cannot be adhered to at this location it is advisable to choose the support conditions for longitudinal forces in such way that both abutments with their generally relatively large stiffnesses take part in carrying the longitudinal forces. For these systems - compare Fig. 4b - where one intermediate span with small substructure stiffnesses will be fixed longitudinally to both corresponsing piers. One pier with only longitudinally sliding bearings may also be considered.

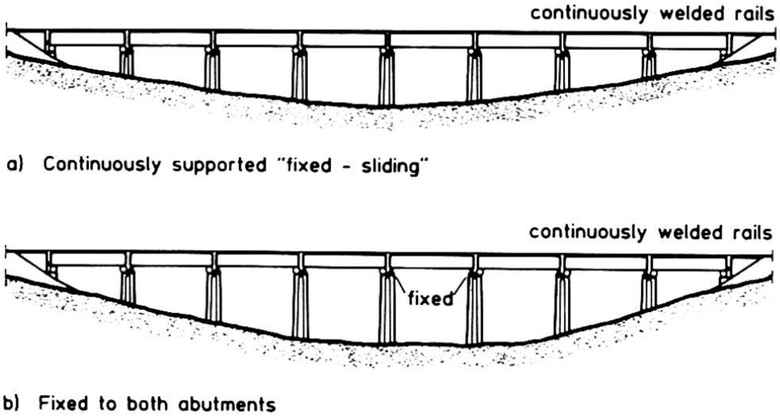


Fig. 4 Simple span valley bridges without special devices

Generally valid limiting values for the applicability of the systems outlined here cannot be given due to their complexity. Calculated examples did show, however, that the permissible stresses and movements can still be adhered to if individual pier stiffnesses amount to only about 40 MN/m which correspond to pier heights of about 30 m.

3.3 Valley Bridge with Special Devices

If the rail stresses or relative movements would become too big for a sequence of simple span girders with continuously welded rails, special devices can be applied. These should permit, if possible, the use of continuously welded rails, as is the case e.g. for the inclusion of creep couplers. These structural elements are located between all simple spans and act only for short-time longitudinal forces. As a consequence they do not react to the changes in length of the structure due to temperature, but carry significant amounts of the braking and acceleration forces to the abutments. Prototypes for hydraulically acting creep couplers are currently in the testing stage.

Another special device connects all single beams by longitudinal force couplers. Hereby, the longitudinal forces are carried similarly as for continuous girders.

4. STRUCTURAL SYSTEMS WITH CONTINUOUS GIRDERS

For continuous girders continuously welded rails are only feasible for short girder lengths so that a rail expension jointing with a discontinuous ballast is the standard detail. By this the participation of the rails in carrying longitudinal forces is strongly reduced. The overall changes in temperature of the structure do not create restraints in the rails as these can only be caused by relative changes in temperature between structure and rails.

Due to the discontinuous rails continuous girders form a straightforward static system for carrying longitudinal forces, which is only very little influenced by the soil stiffness.

The superposition of braking and acceleration forces with the friction forces from the bearings leads to considerably bigger longitudinal forces for continuous girders than for simple span girders. Possible limiting maximum values are:

By full exploitation of the permissible changes in length for the statical rail system, continuous girders with a length of up to 940 m can be built. At a fixed point located at one end, for assumed 58 m spans, friction forces corresponding to Section 2.2 of

$$\max L_F \triangleq 0.035 \times 0.5 \times 915 = 16.0 \text{ MN}$$

may act, onto which braking and acceleration forces of 7.25 MN have to be superimposed. Taking into account the favorable influence of the horizontal bearing stiffness of the abutment and the elastic superstructure elongation, the upper limit reaches in any case a value of

$$\max \sum L > 20 \text{ MN}.$$

This force acts in tension or compression onto the box beam and has to be withstood by the abutment. As these forces are imposed onto other additional loads the tensile forces have to be overcome by prestressing so that from these loads alone compression forces in the range of 40 MN are created.

In order to illustrate the influence of these action forces on a structure they are compared with the prestressing forces required for bending in a 44 m span: at midspan about 32 MN are necessary and at the piers about 45 MN, i.e. an average prestress force of 38 MN. An increase of this prestress by about 50 % would require considerable additional structural measures not compatible with the aim of durable structures. The same holds true for the corresponding abutments. For this reason it is advisable to locate joints and fixed points in such a way that the calculated longitudinal forces do not exceed 12 MN. From this condition the following structural systems offer themselves for carrying the longitudinal forces.

4.1 The Continuous Structure Fixed at one End

In order to permit a relatively simple and quick replacement of a beam by transverse shifting, the lengths of continuous girders for valley bridges of the new DB railway lines are limited to 440 m. Superstructures up to this length are expediently provided with a fixed point at one abutment as shown in Fig. 5. Their longitudinal forces remain within the limits given above and can be carried easily by the abutment. The rail expansion jointing is located at the other abutment, well accessible and hardly influenced by beam deflections in plan and elevation.

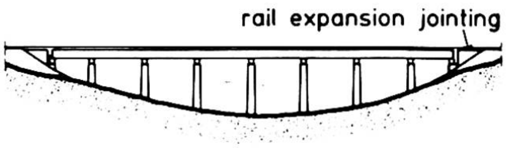


Fig. 5 Continuous girder bridge fixed at one end

4.2 The Continuous Structure with one Central Joint

For bridge lengths between 440 m and 880 m it is advisable to repeat the continuous structure mirror image as shown in Fig. 6. The same conditions for each part are created in this way with regard to longitudinal forces and their transmission. The rail expansion jointing for movements up to 800 mm is now located at the transition pier.

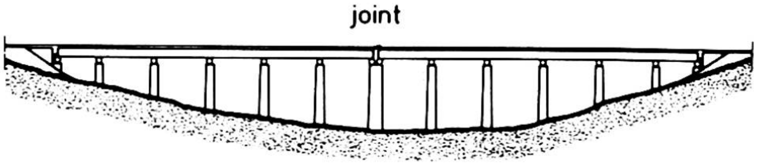


Fig. 6 Continuous girder bridge with one central joint

4.3 Structures with Hydraulic Longitudinal Bearings

For bridges longer than discussed above it is not possible within the framework of the given parameters to use exclusively the abutments as fixed points. Traditional piers are only in a very limited way suitable for the location of fixed points of railway bridges because the large braking and acceleration loads act partially dynamically. In the most unfavorable case the braking jolt may cause uncomfortable oscillations. Such a load transmission would in any case cause additional deformations and movements. Special devices between piers for carrying the longitudinal forces with small deformations require generally quite an effort.

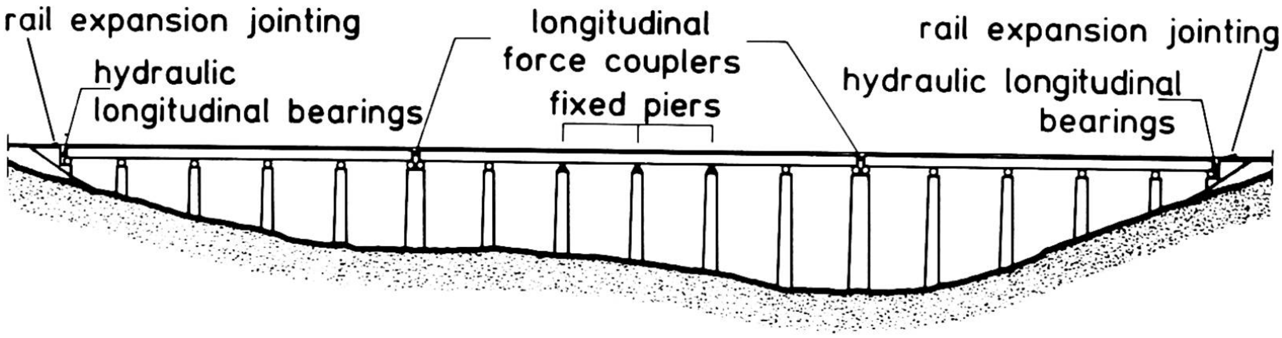


Fig. 7 Continuous girder bridge with hydraulic longitudinal bearings

On this background and in order to achieve as small as possible longitudinal forces acting in tension the system shown in Fig. 7 proves especially advantageous: Joints are located at both abutments, and in the bridge center one or more piers are fixed to the superstructure. This group of fixed points has the sole purpose of carrying the differntial friction forces from both bridge halves. Braking and acceleration loads are transmitted to the abutments by means of hydraulic bumpers acting in compression. With two rail expansion joints for movements up to 800 mm on the abutments this system can be built for bridge lengths up to 1600 m. In this case the biggest tensile force still remains below 12 MN and compression forces only up to 7.25 MN act on each abutment. This system offers itself, of course, for bridge lengths below 880 m also.

In order to permit replacement of the structure sectionswise, joints with longitudinal force couplers are provided.

The hydraulic longitudinal bearings are very similar to the hydraulic creep couplers in design and action. In difference to those, however, they do not have to fulfill special deformation conditions. Maintenance requirements and lifetime are estimated to be similar as for other bearings. Positive experiences are available from various 10 to 15 year old bridges.

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Concrete Track Systems for Maglev Vehicles

Voies en béton pour véhicules à sustentation magnétique

Betonfahrwege für Magnetschwebefahrzeuge

Herbert SCHAMBECK
Dipl.-Ing.
Dyckerhoff & Widmann AG
Munich, FR Germany

Herbert Schambeck, born in 1927, graduated in civil engineering at the Munich Technical University in 1949. Employed by Dyckerhoff & Widmann AG since 1950, he was appointed Director in the headquarters of this company in 1974. He designed widespan industrial buildings and prestressed concrete bridges. Numerous lectures and publications on prestressed concrete Bridges.

SUMMARY
A report is given on the experiences gained in the Fed. Rep. of Germany in the design, construction and operation of concrete track systems for magnetic levitation vehicles. The largest project of this kind is the Emsland Transrapid Test Facility (TVE), whose elevated track system will be approx. 31.5 km long when completed and on which tests will be carried out at speeds of up to 400 km/h.

RESUME
Le rapport fait le point de l'expérience acquise en République Fédérale d'Allemagne dans la conception, l'étude, la construction et l'exploitation de voies en béton pour véhicules à sustentation magnétique. Le projet le plus important est l'aménagement de l'installation expérimentale de la région d'Ems (Transrapid Versuchsanlage Emsland, TVE): une fois terminée, la voie surélevée aura 31,5 km de long et permettra de réaliser des essais de grande vitesse à 400 km/h.

ZUSAMMENFASSUNG
Es wird über die in der Bundesrep. Deutschland bei Planung, Bau und Betrieb von Betonfahrwegen für Magnetschwebefahrzeuge gesammelten Erfahrungen berichtet. Das grösste Vorhaben dieser Art ist die Transrapid Versuchsanlage Emsland (TVE), deren aufgeständerter Fahrweg im Endausbau ca. 31,5 km lang sein wird und auf der Geschwindigkeiten bis zu 400 km/h erprobt werden sollen.

1. GENERAL NOTES

Development of the magnetic levitation technology has been pursued in Germany since 1970 by industry and supported by the Federal Minister of Research and Technology. Initially, work was performed in parallel on the electromagnetic and the electrodynamic levitation technology. A system decision between the two ended in favour of the electromagnetic technology with the two alternatives short stator and long stator as motor. In 1978 the Federal Minister of Reserach and Technology (BMFT) gave his approval for the design and construction of a test facility in Emsland by a consortium of 7 German companies for a system with long stator (Emsland Transrapid Test Facility = TVE). Essential component of this facility is a test section which when completed will be approx. 31.5 km long and on which speeds up to 400 km/h are to be tested /1/.

From the very start of the development of magnetic levitation systems, development of appropriate and economical track systems was pursued in the German construction industry and supported by the BMFT because

- the requirements placed on the track system differ from those for conventional transport systems,
- the costs for the track constitute a very large portion of the overall costs of a new transport system and
- practicable, optimized track structures are a prerequisite for the feasibility of a new transport system.

Track systems for a later operating section will be 1 or 2 track and will be routed on the surface, elevated or in tunnels. Planning work accomplished to date has demonstrated that of these three levels, the elevated track system has the greatest significance because

- through the elevation system, use of the land below the track is practically not obstructed, and this will make it easier to get a new section accepted,
- the loads of magnetic track vehicles are relatively small and accordingly the construction costs of an elevated system are reasonable,
- the vehicle clearance with exterior embracing of the track girder is designed in such a way that the difference between the construction of not elevated and that of an elevated track system is not so significant as with conventional systems,
- the aerodynamical problems connected to high speeds are best solved with an elevated track system. The track equipment is protected against vandalism.
- In addition, the alignment elements (radiuses and gradients) are so favourable that the track can to a large extent follow the shape of the land. The resulting uniformity of the structure produces a large rationalization effect.

For these reasons, the track system in Emsland is an elevated structure.

The elevated track can be built in concrete or steel. Tender results with binding bids of competing companies have led to the fact that of the 20.5 km long first construction section of the TVE so far completed, the entire substructure (foundation und columns) as well as approx. 15.5. km of the superstructure have been built in concrete and approx. 5.0 km of superstructure in steel. The test section will be completed in 1985 with an additional approx. 11 km of a second construction section. The design for the second section is compatible with that of the first section, but experiences gained to date have been taken into consideration (Fig. 1).

2. REQUIREMENTS PLACED ON THE TVE TRACK SYSTEM

2.1 Vehicle clearance (Fig. 2)

The vehicle with exterior embarcing of the tracks leads to a single-girder system. From the point of view of structural engineering this is superior to a double girder with interior embracing (erected for a test facility built in 1972). The girder dimensions and the possibilities for location of cross girders at the supports are influenced largely by the clearance of the vehicle.

2.2 Equipment of the track system (Fig. 2)

It forms the functional areas for the vehicle. They are

- the stator armature (sa), which in normal operation bears and drives the vehicle,
- the lateral guide rails (lgr) for lateral guidance of the vehicle and
- the sliding skids (ss) in case of lowering of the vehicle when the bearing magnets fail.

The fastening of the equipment to the carcass structure is an important planning element.

2.3 The loads (Fig. 3)

The critical values of the resultant forces from the live loads (ll_1 and ll_2) alone and from the sum of live loads and dead loads of the concrete girder ($dl + ll_1$ and $dl + ll_2$) are entered in Figure 3 for the curve travel with a transverse slope of 12°. The diagramm shows that a high dead load acts favourably in this case, since as a result the eccentricity of the resultants from the total loads is reduced and alternating stresses from live loads are decreased.

2.4 Alignment elements

The radiuses in plan and elevation, the gradient, the track twisting and the crossfall are fixed by the alignment. They must be taken into adequate consideration in the track design.

With TVE, the smallest horizontal radius was planned for 1 000 m, the smallest vertical radius for 8 000 m, the largest longitudinal gradient at 3.5 % and the largest crossfall at 12°, the greatest twisting (= change of the crossfall per lin m) is 0.08°/m.

2.5 Rigidity requirements

Vehicle and track represent a coupled oscillation system, due to which specific character-

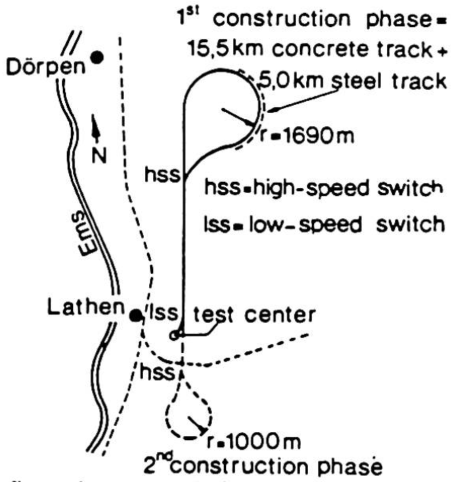

figure 1

The Emsland Transrapid Test Facility (TVE)

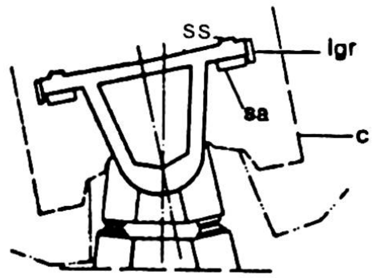

figure 2
Vehicle clearance and equipment of the track

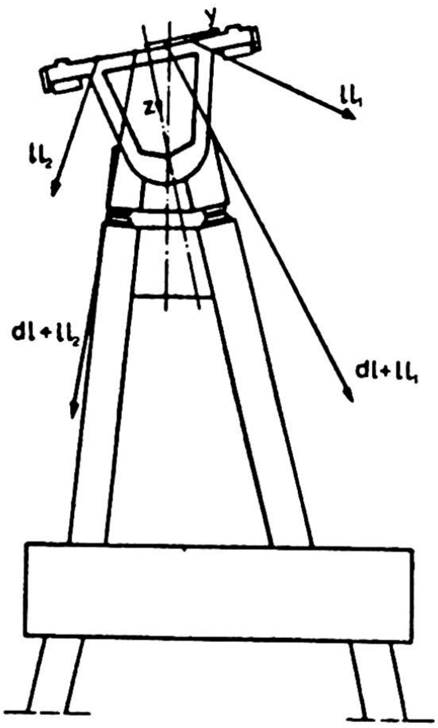

figure 3
loads of the Transrapid test track

istic frequencies and rigidities must be required from the track based on extensive tests and experiments. Here it is critical in particular when the vehicle hovers above the track at low speed or at standstill - that is when starting and stopping. In simplified form it can be stated that these requirements have been fulfilled in the previously built span range of concrete structures when they are designed and dimensioned as prestressed concrete hollow box girders in compliance with the regulations prevailing in bridge building /2, 3/.

2.6 Tolerances

The gap width between the magnets of the vehicle and of the track is as a regular case 10 mm. The permissible deviation from this standard gap width and accordingly the permissible deviation of the f u n c t i o n a l a r e a s from their nominal position are only a few mm. These requirements are considerably more stringent than previously applied in the construction industry; they exercise considerable influence on the design and construction method for the track system.

Specifically, a distinction must be made between

long-wave dimensional devitations in x, y and z direction (see Fig. 3),

short-wave dimensional deviations in x, y and z direction,

dimensional deviations at the joints of the girder ends.

The permissible total tolerances of the functional areas are made up of various influences, namely

inaccuracies in the fabrication and installation of the equipment,

deformations resulting from temperature changes in the girder,

long-term deformations of the concrete girder under permanent loads,

substructure settlements.

In addition, elastic deformations of the girder under live loads are subject to special limitations.

The tolerances of the c a r c a s s s t r u c t u r e must be regarded separately from the tolerances of the functional areas. They, too, are considerably smaller than previously practiced in the construction industry. In essence, we differentiate again between the influences stated above. Accordingly the inaccuracies of the carcass structure due to construction must remain small because the existing thin girder structure allows only a few mm of deviations between position of carcass structure and equipment. With respect to deformations, the requirements placed on the carcass structure are more or less identical with the requirements imposed on the functional areas.

2.7 Creative requirements

In the world of today, a new tranport system can be realized only if it keeps the associated environmental damages within acceptable limits. One prerequisite for this is that the track system is well designed architecturally. The accompanying picture (Fig. 4) shows that great importance was attached to this aspect for the TVE track system.

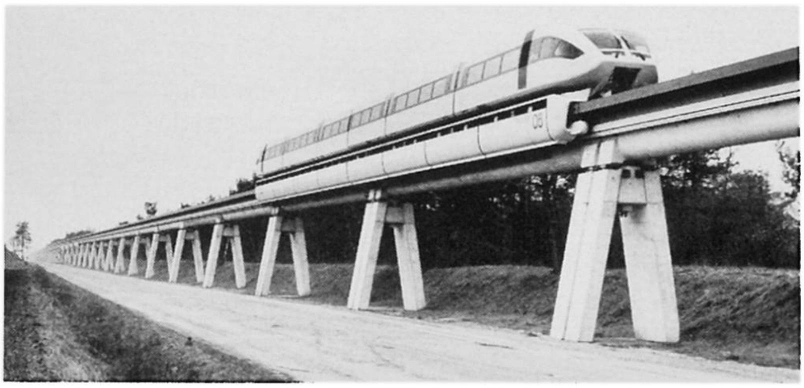

Fig. 4

2.8 Maintenance

The track system, the equipment and their connections must be designed in such a way that a long service life is guaranteed and that the costs for later maintenance are low.

In summary, it can be stated that the requirements placed on the track system - are extremely strict and will produce additional costs in comparison to the otherwise applicable standard in the construction industry. Greater dimensional inaccuracies resulting in a larger gap width between the magnets would consequently lead to higher costs with respect to the vehicle, with respect to system operation and with respect to reduced pay-load capacity. Is was the objective of the previous development to establish the requirements placed on the track system that they represent a cost minimum for the overall system.

3. STRUCTURAL CONCEPT OF THE TRACK SYSTEM

The concrete track of the TVE represents a practical and rational design for the given requirements. Its essential features are described and substantiated in the following.

Concrete as building material for the substructure and superstructure provides high dead load and accordingly low load eccentricities in the foundation,

provides high rigidity and good vibration absorbing characteristics and consequently insensitivity to vibrations,

provides good acoustical insulation,

is economical in cost,

is available in practically all countries and

with professional planning, execution and supervision guarantees low maintenance costs and long operational life.

Tensioning of the concrete is necessary to exclude plastic deformations under permanent loads completely or at least to a large degree. The problem of plastic deformations cannot be solved with reinforced concrete.

The accuracy of the deformation behaviour of the girder is increased by using tendons without relaxation and with very low slippage in the anchorages, by applying an exactly defined concrete mixture and by avoiding conctruction joints and other inhomogeneities within the girder.

Large precast elements (i.e. jointless production of the entire cross-section over span length) permit

mechanization of the construction operations,

minimization of costs,

high construction speed regardless of weather and

high accuracy and material quality.

Single-span girders were selected because the deformation conditions prevailing with the TVE could be fulfilled with single-span girders in post-tensioned concrete and because assembly and disassembly are simple.

The standard span selected was approx. 25 m with a structural height of 1.80 m and approx. 31 m with a structural height of 2.40 m; special spans of approx. 37 m were executed in cast in situ concrete.

A box section for the superstructure produces great bending rigidity in z and y direction and great torsional rigidity (Fig. 3).

Simple compression bearings with prestressing by DYWIDAG tensioning bars were used both for transferring the resultants from the vertical forces and the transversely directed horizontal forces (type 1 according to Fig. 5) and for transferring the brake forces (type 2). They are easy to install, require almost no maintenance and are economically priced. The bearings of type 1 are spread in transverse direction as far as the vehicle clearance permits. In the longitudinal section, they are located as close as possible to the girder end, because vertical movement of the functional areas at the joint between two girders should be as small as possible.

Columns designed as an A-frame transfer the large horizontal forces of the superstructure safely into the foundations.

A pile foundation assures minimum foundation settlements despite poor subsoil conditions existing in Emsland.

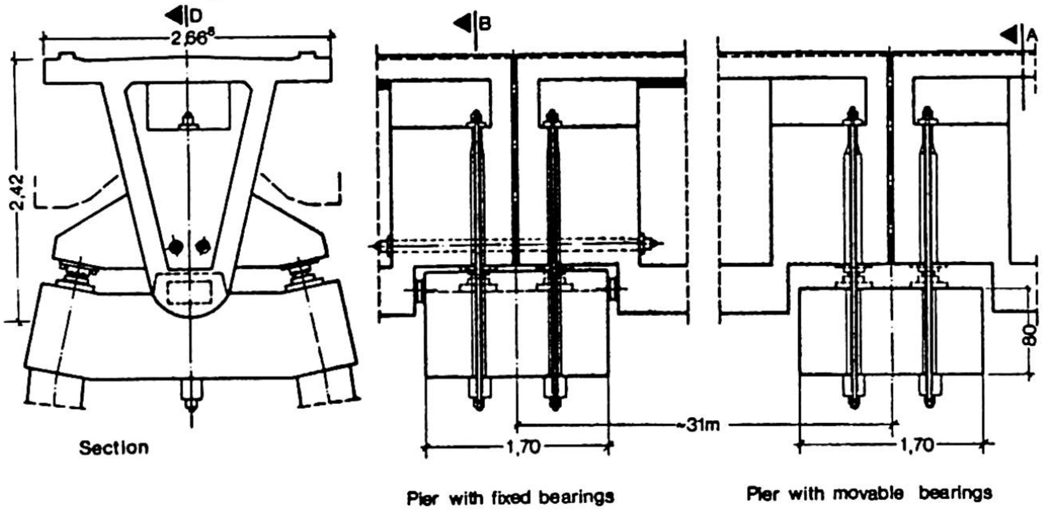

figure 5
Bearings of the 31m girder in the 2nd construction section

Separation of the production of the carcass structure and the equipment was consistently maintained to permit greater production tolerances in the carcass construction than in the functional areas and to assure that the initial deformations of the concrete do not influence the accuracy of the functional areas'position. There are 2 different methods:

With the first method, the equipment is assembled with the aid of a placing train and supplementary facilities after the track girder has been finely adjusted on the columns. The method has the advantage that the dimensional tolerances of the functional areas are not influenced by the inaccuracies in adjusting the carcass structure. It has the disadvantage that tolerances for deformations of the girder under the load of the placing train and as a result of temperature conditions must be taken into consideration during assembly of the equipment.

With the second method, the equipment is assembled in a shop under controlled temperatures and with a device which does not load the girder before placing of the girder. The equipment assembly is not dependent on the weather. The tolerances for the fine adjustment of the completely equipped girders now influence the tolerances of the functional areas in operating condition.

Fastening the stator armature and lateral guide rails to the concrete girder can be made according to two different systems.

In the first system, recesses are provided in the concrete girder, into which fastening devices for the stator armature and the lateral guide rails are subsequently inserted and grouted with fast-setting mortar.

In the second system, steel elements are cast into the girder during production of the girder. Threads are then drilled into these steel elements immediately before placing the equipment.

The sliding skids on top of the girder are produced together with the remaining structural concrete - that is initially with larger tolerances - and later grinded down to exact nominal position.

Adjustment possibilities are provided by bearings which with the aid of hydraulic jacks and by insertion of washer plates can be adjusted in all 3 directions.

In addition, tendons without bond can later be installed inside the girder box, allowing to control deformation of the girder through a freely selectible tensioning force.

4. CONSTRUCTION PERFORMANCE

For the Emsland Transrapid Test Facility (TVE), cast in situ concrete proved to be the most practical and most economical solution for the foundations and the piers including the pier heads /4/.

Parallel to the production of the substructures, production of the track girders was carried out in a field plant in the vicinity of the project site.

The first step in the girder production was the prefabrication of the reinforcing cage (RC) in a separate work shop (1).

The reinforcing cage already contained the recoverable interior formwork (IF) shaping the inside of the box girder. The reinforcing cage was then transported into the concreting work shop where it was lifted into the exterior formwork (EF) (2).

Three exterior formworks were available. After overcoming the initial period for starting construction, one standard girder was cast per day in each exterior formwork (3).

Following a heat treatment of the newly cast concrete, a partial posttensioning was applied as early as 16 hours after concreting so that the girder could be lifted out of the formwork and placed on the storage yard (4).

After final posttensioning, after installation of transverse diaphragms at the girder ends and after at least 2 months of storage for fading of initial deformations due to creep and shrinkage, the girder was transported with heavy lorries to the installation site (5).

The girders were initially placed with cranes onto temporary bearings and later precisely positioned in an independent operation; special hydraulic jacks were used for this purpose which permitted movements in all directions and which carried the girder until the bearings had been finally adjusted.

The stator armature and the lateral guide rails were then installed in sections of about 12,5 m length using the above described method 1 with a placing train. The sliding skids were brought to their final level with the aid of the grinding vehicle.

All construction work was accompanied by a program of extensive and expensive surveying work.

The above described construction method is adapted to the given project - that is building approx. 15 km of track in well accessible terrain. For larger track lengths in difficult terrain, an alternative placement method was developed moving precast elements over the previosly placed girders.

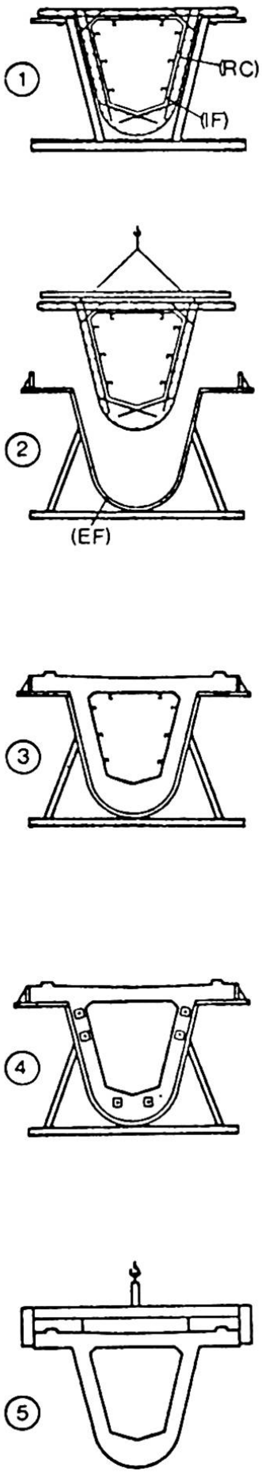

figure 6
Production of the 25m girder as prestressed concrete precast element

5. CHARACTERISTICS OF THE FINISHED TRACK

Adherence to the tolerances, shapes and dynamic properties required in the specifications was checked by various measurements.

Deformation measurements under static load showed that the actual rigidity of the girders is somewhat greater than that required and that the deviation range of these measurements is very small.

Long-term measurements were made both before and after installation of the equipment on the girder. The measurements taken before installation of the equipment were used for the definitive fixing of the vertical position of the equipment at the time of installation. Measurements made after installation of the equipment confirmed that in this condition the sustained exterior loads and the deflection forces of the tendons are optimally matched to each other and that therefore the plastic deformations are almost zero up until now.

Temperature measurements likewise produced the result that the temperature differences between the top and bottom of the girder assumed in the calculation have in actual practice not been exceeded.

The acceptance measurements for checking the position accuracy of the equipment were initially made section by section and thereafter continuously by means of a test vehicle. The vehicle itself - which is 54,2 m long, consists of 2 units, can accommodate 98 persons per unit and is called TR 06 - is also equipped with a measuring device, with which the position accuracy of the functional areas can be recorded during the test runs.

Dynamic tests for determination of the lowest inherent frequencies, of the accompanying inherent deformations and of the absorber capacity showed that the natural frequency of the basic oscillation is 5.8 Hz for the 31 m girder and thus comes very close to the calculated value. The absorber capacity - expressed by the logarithmic decrement of successive amplitudes in the fading process - can be assumed at Λ = 0.1 to 0.2. The relatively high absorber capacity compared to known values for bridges appears to be due to the fact that the foundation is also participating in the vibrations.

6. PARTIES INVOLVED

The Transrapid Test Facility Emsland (TVE) has been designed and executed by a consortium of the companies AEG, BBC, Dyckerhoff & Widmann, Krauss-Maffei, MBB, Siemens, Thyssen Henschel and is sponsored by the Federal Minister of Research and Technology. The parts of the test facility constructed in concrete have been designed and erected by Dyckerhoff & Widmann AG assisted by other German construction companies. Once completed, the test facility will be taken over by the newly established company MVP (constituted by representatives of Deutsche Bundesbahn, Lufthansa and IABG) for experimental operation.

REFERENCES

/1/ HILLIGES, D., MÖLZER, P., RASCHBICHLER, H.-G. and ZUREK, R.: Der Fahrweg der Transrapid Versuchsanlage Emsland, ZEV-Glas.-Ann. 105 (1981), No. 7/8

/2/ HILLIGES, D.: Fahrwege für Magnetschwebefahrzeuge unter dynamischer Beanspruchung, Bauingenieur 51 (1976), p. 337-347

/3/ HILLIGES, D.: Dynamik von Magnetbahnfahrwegen in Betonbauweise, Meßergebnisse und ihr Einfluß auf den Entwurf, VDI-Reports No. 510, March 1984, p. 107-113

/4/ LÖNNECKE, K. H. and STÜBEN, H. H.: Bauausführung des Betonfahrweges der Transrapid Versuchsanlage Emsland, Bauingenieur 58 (1983), No. 4

/5/ Inst. f. Building Materials, Solid Construction and Fire Protection at the Braunschweig Technical University: Schwingungsmessungen am Betonfahrweg der Transrapid Versuchsanlage Emsland, Untersuchungsbericht No. 83 063 -Pm/br, 1.2.1983

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).

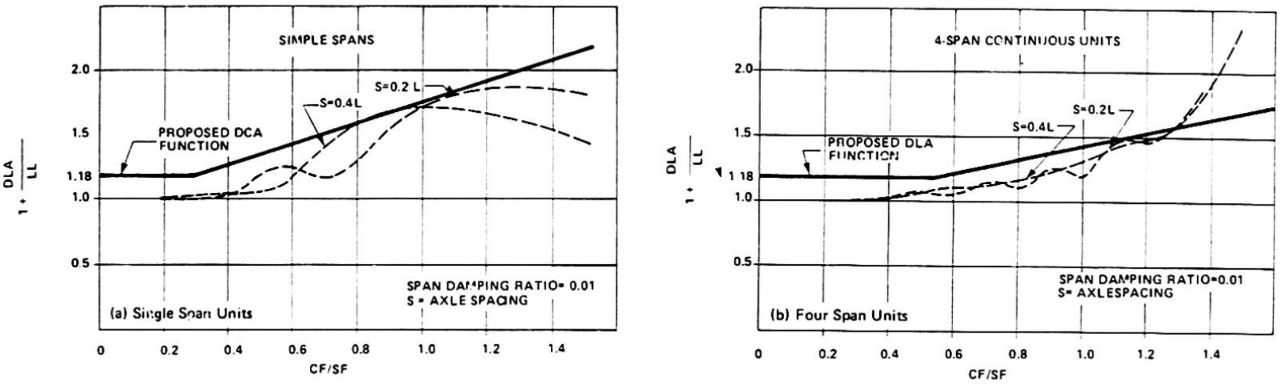

Figure 1/ Dynamic Load Allowance as a Function of the Crossing Frequency Ratio CF/SF

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.

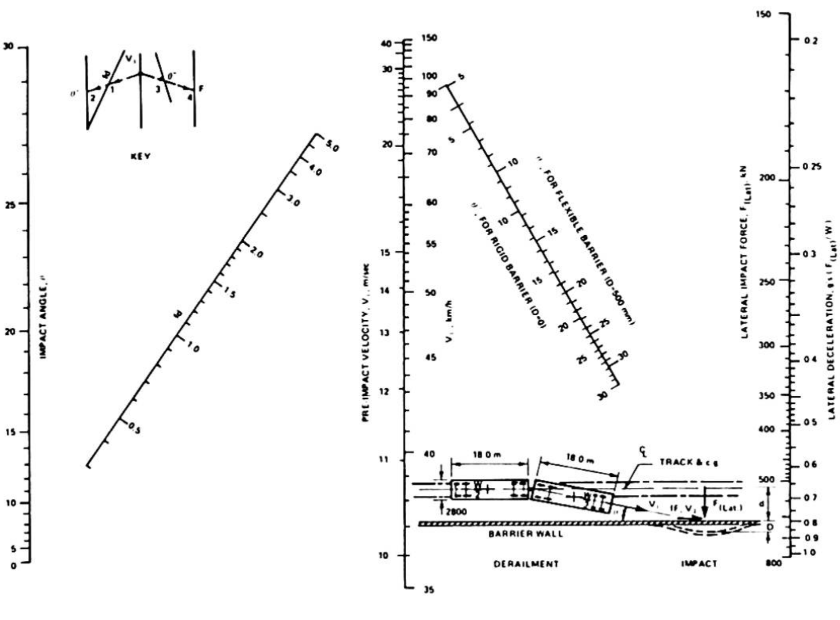

Figure 2/ Vehicle Mis-hap Force Effects

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.

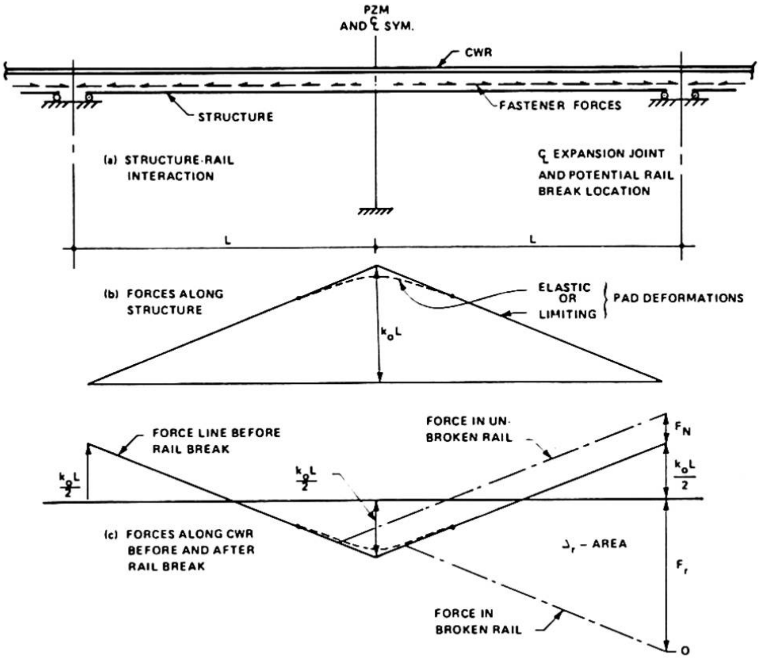

Figure 3/ Structure-rail Thermal Interaction

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:

$$\Delta_r = \frac{F_{r(max)}^{\,2}}{2k_oA_rE_r}, \text{ for stiff fasteners where, } \frac{F_r}{k_oL} < 1.5$$

and, $$\Delta_r = \frac{F_r^{\,2}}{2k_oA_rE_r}, \text{ for flexible fasteners where, } \frac{F_r}{k_oL} > 1.5$$

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:

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

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:

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

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

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:

$$F_N = \frac{\Delta F}{\dfrac{K_cL}{A_rE_r} + N}$$

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

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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.

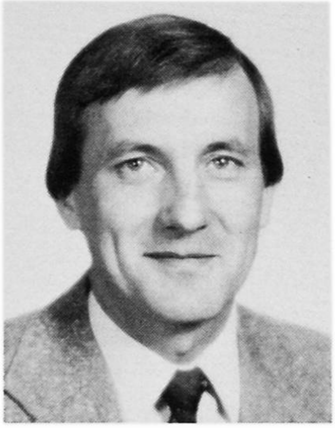
[11] FISHER, J.W., Bridge Fatigue Guide -- Design and Details, American Institute of Steel Construction, 1977, p. 58.

Prestressed Concrete Ties for Elevated Rail Transit Structures

Tirants en béton précontraint pour voies ferrées suspendues

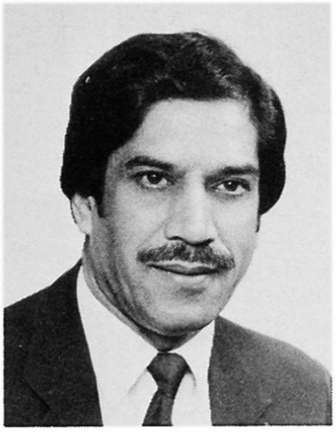
Vorgespannte Betonschwellen für Hochbahn-Tragsysteme

T.I. CAMPBELL
Prof. of Civil Eng.
Queen's University
Kingston, Canada



Ivan Campbell, born 1940, received his engineering degrees from the Queen's University of Belfast, N. Ireland. For the past fifteen years he has been involved in research into prestressed concrete transportation structures.

M.S. MIRZA
Prof. of Civil Eng.
McGill University
Montreal, Canada



Saeed Mirza, a graduate of the University of Karachi, Pakistan obtained his M.Eng. and Ph.D. degrees at McGill University, Montreal, Canada. He has been active in research into the behaviour of structural concrete elements and subsystems using physical and computer (finite element) models over the past twenty years.

SUMMARY
A study of the distribution of wheel loads among the ties in an open deck railway bridge containing precast prestressed concrete ties is described. Analytical modelling of the system indicated that only three parameters have a significant influence on load distribution. Full scale load tests confirmed the findings of the analytical model. An open deck system may find application in elevated guideways for transit vehicles.

RESUME
Une étude sur la distribution des charges d'essieux sur les tirants d'un pont de chemin de fer à tablier suspendu est présentée. L'étude analytique a montré que seuls trois paramètres ont une importance significative, résultat confirmé par les essais. Ce genre d'ouvrage peut être utilisé pour des voies de chemin de fer suburbaines surélevées.

ZUSAMMENFASSUNG
Eine Studie über die Verteilung der Radlasten auf die direkt auf den Stahllängsträgern aufliegenden vorgespannten Betonschwellen einer offenen Bahnbrücke wird präsentiert. Die analytische Modellierung des Systems zeigte, dass nur drei Parameter einen bedeutenden Einfluss auf die Lastverteilung haben. Belastungsversuche bestätigen die theoretischen Ergebnisse. Offene Fahrbahnsysteme können ihre Anwendung in Hochbahn-Tragstrukturen finden.

1. INTRODUCTION

Open decks are widely used for bridges on the railway systems in Canada. Such a bridge comprises rails supported on ties spanning between two main longitudinal girders. Unlike a conventional railroad bridge where the ties are supported by ballast, the ties transfer the wheel loads directly from the rails to the main longitudinal supporting girders and thus are referred to as bridge ties. This structural system is economical in that the dead weight of ballast is eliminated.

A deck with timber ties is susceptible to fire, which not only causes damage to the timber ties but also to other components of the bridge superstructure, resulting in major traffic disruptions and hazards to life and property, as well as in costly maintenance. Consequently, the Canadian Railways (Canadian National and Canadian Pacific) are considering replacing the timber ties with precast prestressed concrete ties.

An open deck bridge system with concrete ties is shown in Fig.1. The arrangement is similar to the one with timber ties, except that elastomeric pads are incorporated between the rails and the ties (rail-tie pad), and between the ties and the supporting girders (tie-girder pad). These pads are required since a concrete tie is much stiffer than a timber tie and a larger impact would result from wheel loads if the pads were not incorporated.

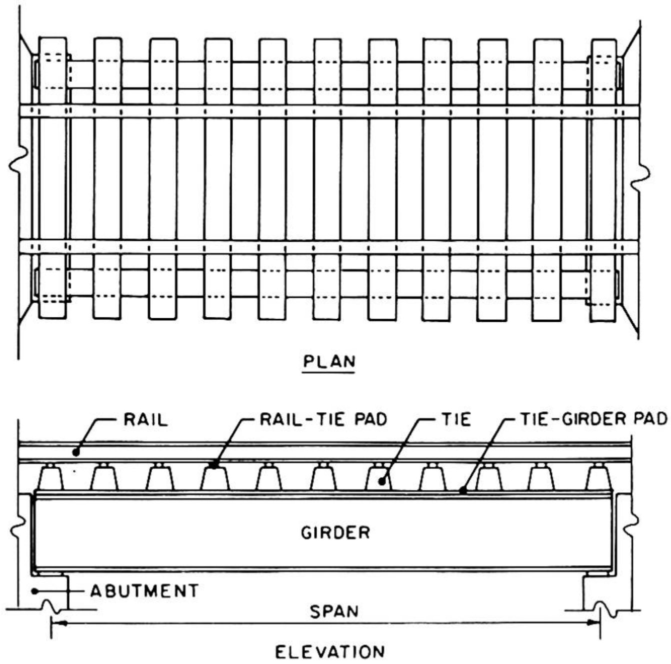


Fig.1. Open deck railway bridge with concrete ties

A recent research program on precast prestressed concrete bridge ties, has been carried out jointly by McGill University, Montreal, Canada [1] and Queen's University, Kingston, Canada [2]. This program studied the distribution of wheel loads to the ties in an open deck system with precast prestressed concrete ties, as well as the strength of the ties under static and repeated loadings. The distribution of wheel loads among the ties was studied by both analytical models and full scale laboratory testing and is described in this paper.

The analytical work on wheel load distribution showed that the distribution of a wheel load among the ties is affected mainly by the stiffness of the tie-girder pads, the type of rail and the spacing of the ties. It was also found that, for a particular type of rail and stiffness of tie-girder pad, the percentage of wheel load taken by a tie increased linearly with tie spacing. The validity of the analytical model was confirmed by the full scale load testing. The load tests also showed that improper seating of the ties on the tie-girder pads, as a result of differential elevation at the rail seat of the ties prior to installation of the rail, influenced the distribution of wheel loads significantly. Assuming proper seating of the ties, the maximum load taken by a tie is about one-third of the wheel load when soft tie-girder pads are used. However, the load level in a tie can rise to around 60 percent of the wheel load when the ties are improperly seated. Based on this research program, recommendations with regard to suitable design criteria for a precast prestressed concrete bridge tie were made. These recommendations are being evaluated by the Canadian Railways in a field appraisal program.

Although this system has been developed for conventional railway track, it is felt that a similar system could be utilized on elevated guideways for rapid transit vehicles, particularly in non-urban environments where aesthetics and noise level are not critical. The findings of this study would be applicable to such a system.

2. THEORETICAL STUDY OF LOAD DISTRIBUTION

The distribution of a wheel load to the ties depends on the properties of the various components of the system (see Fig.1), namely:

(a) location of the wheel load with respect to the ties;
(b) stiffness of rail-tie pads;
(c) differential elevation of the ties;
(d) stiffness of the supporting girders;
(e) type of tie;
(f) spacing of supporting girders;
(g) stiffness of tie-girder pads;
(h) type of rail;
(i) spacing of ties.

The relative effects of these variables were established from a parametric study using an analytical model of the system.

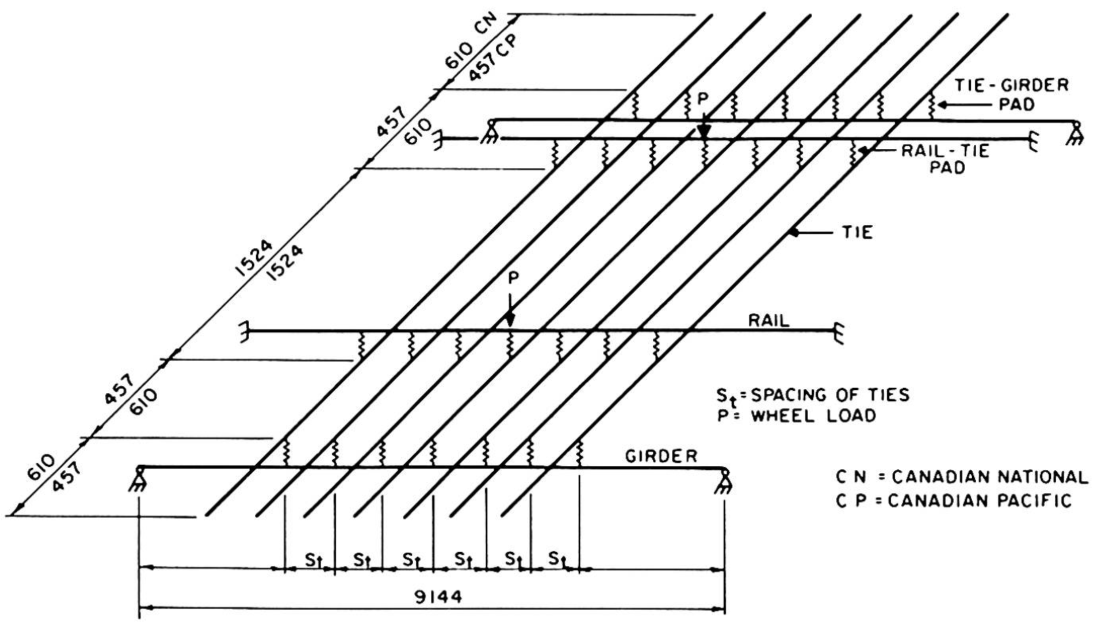


Fig.2. Idealized bridge system

An idealization of a typical system is shown in Fig.2. It was assumed that the main supporting steel girders were simply supported over a representative span of 9.1 m. The length of the rails was taken as the same as the bridge span and both ends of each rail were assumed to be fixed to simulate continuity of the rail. Both the rail-tie pads and the tie-girder pads were modelled as linear elastic springs. The computer program SAP IV [3] was used for the analysis. The rails and bridge girders were simulated by beam elements, while the pads were simulated by truss elements, whose properties were chosen to give the required stiffnesses. As shown in Fig.2, a wheel load was applied simultaneously to each rail.

Table 1 Variation of Parameters of the System

Parameter	Unit	Value			
Stiffness of rail-tie pad	kN/mm	315	Rigid	—	—
Stiffness of girder (EI)	$MN.m^2$	416	830	1664	—
Type of tie	-	CN`A'	CN`B'	CP	—
Spacing of girders	m	2.44	2.74	—	—
Stiffness of tie-girder pad (K)	kN/mm	25	50	99	180
Type of rail	kg/m	49.6	57.0	65.5	—
Spacing of ties (S_t)	mm	406	457	508	559

Initially models containing three, five, seven and eleven ties were studied. In each case the wheel loads were positioned over the central tie since this represents the most critical condition of loading in the central tie. It was concluded that the model with seven ties was adequate, since the load distributions among the ties for the seven and the eleven tie arrangements were similar. Further, with the eleven tie system, the outermost ties were subjected to uplift. Also, it was found that the load distribution was the same whether the rail-tie pad was assumed to be rigid or to have its actual stiffness of 315 kN/mm. Consequently, a model incorporating seven ties and rigid rail-tie pads was used throughout the study. The variations considered for the parameters of the system are shown in Table 1. The rationale used in establishing these values and details of the type of tie are given in References 1 and 2.

Figure 3 shows the distribution of the wheel loads among the seven ties for three different girder stiffnesses where girder stiffness has been defined as the product of the modulus of elasticity (E) and second moment of area (I). The load, expressed as a percentage of a wheel load, refers to the load which each rail exerts on the tie. The values of the other parameters of the system are indicated in Fig.3. It is seen that the maximum load occurs in the central tie (tie number

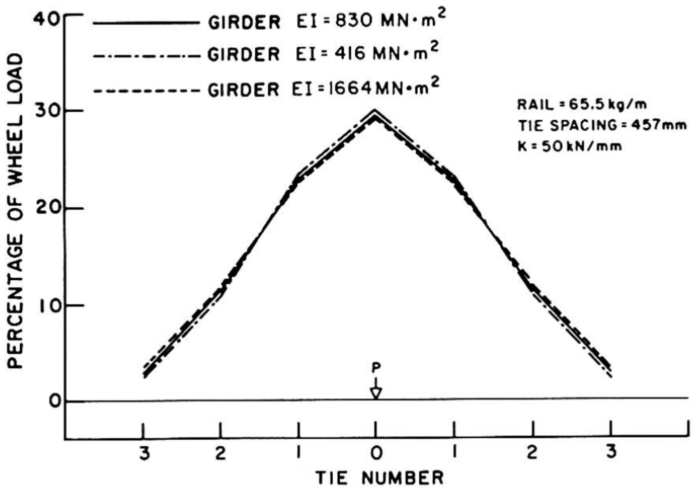


Fig.3. Effect of girder stiffness on load distribution

'0'), which is located underneath the wheels, and that the distribution among the ties is similar for the three different girder stiffnesses. It may be concluded that the stiffness of the supporting girders is not a significant parameter as far as load distribution is concerned. A similar conclusion was reached regarding the effect of the type of tie and the spacing of the girders.

Figure 4 shows the distribution of wheel load among the ties for a particular system in which the stiffness (K) of the tie-girder pads varies as shown. It is seen that the distribution of load is influenced by variations in the tie-girder pad stiffness. The load taken by the tie under the wheel increases with pad stiffness. It may be concluded that the stiffness of the tie-girder pad is a significant parameter for load distribution. A similar conclusion was reached regarding the type of rail and the spacing of the ties. Thus only these three parameters influence load distribution significantly.

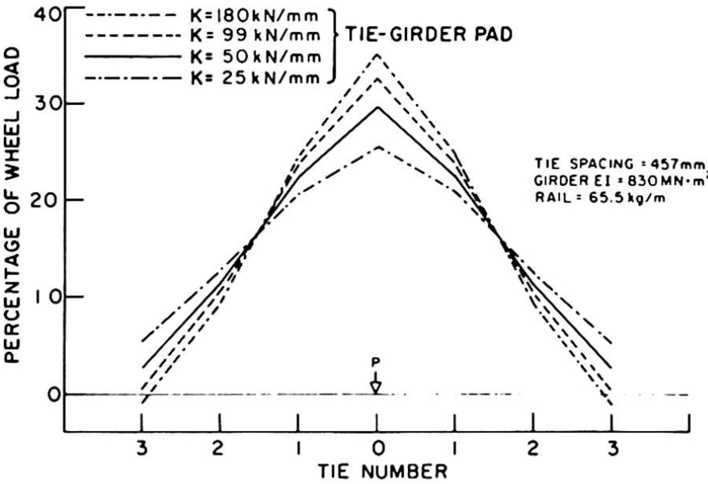


Fig.4. Effect of tie-girder pad stiffness on load distribution

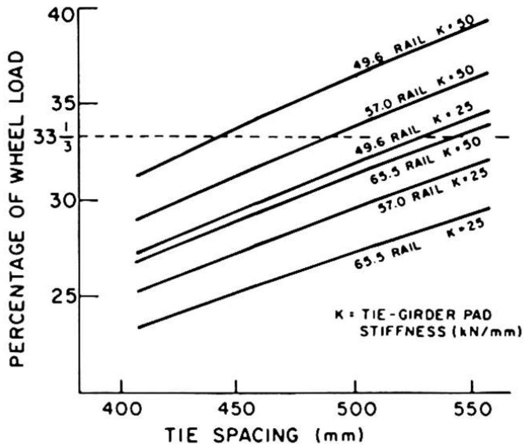


Fig.5. Variation of tie loading with tie spacing, rail type and tie girder pad stiffness

Figure 5 shows the variation of load in the tie under the wheel loads with tie spacing for different types of rail and tie-girder pad stiffnesses. The load increases with tie spacing approximately linearly in each case. Consequently, an equation having the following form may be used in estimating the maximum wheel load in a concrete bridge tie:

$$D = A + BS_t \qquad (1)$$

where D is the percentage of wheel load on the bridge tie and S_t is the tie spacing (mm), while A and B are constants given in Table 2.

Table 2 Constants A, B for Equation 1

Type of rail (kg/m) \ Pad stiffness (K) (kN/mm)		25	50
49.6	A	8	10
	B	0.048	0.053
57.0	A	7.5	9
	B	0.044	0.050
65.5	A	7	8
	B	0.041	0.047

It can be seen from Fig.5, that the commonly used design assumption [4] where the maximum load taken by a tie is equal to one-third the wheel load is reasonable, provided relatively soft tie-girder pads are incorporated in the system.

The analytical model assumes that all the ties are properly seated on the tie-girder pads. During the laboratory load testing it became apparent that improper seating could occur due to variation in elevation at the rail seats of adjacent ties and subsequent lifting of ties from the tie-girder pads when the rail was installed. This variation in elevation results from variation in the thickness and from differential camber of the ties. The effects on load distribution of improper seating of the ties was investigated using the analytical model by assuming gaps to exist between the ties and the girders at various locations. Figure 6 shows the influence of a gap between the girder and one end of the tie under the wheel loads. The distribution for the system without gaps is shown for comparison. It is seen that, in the central tie, the load at rail A, which is the rail adjacent to the end of the tie where the gap exists, is an uplift (tension in the rail fastener), while the load at the other rail (rail B) is higher than that when no gap exists.

Overall, it was found that the presence of gaps in the system resulted in loads in some of the ties being higher than those in the equivalent system without gaps. In a system with a single gap, the highest percentage of wheel load (41%) was found to occur in the central tie when the gap existed in the adjacent tie. From the study of a system with two gaps, a maximum load of 57% of a wheel load was found in the central tie when a gap existed at one end of both the ties adjacent to the central tie.

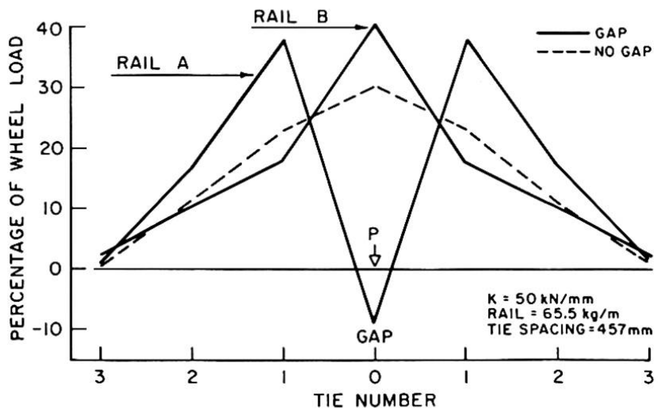


Fig.6. Effect on load distribution of a gap at the central tie

When the differential elevation effect is considered, the design assumption that the maximum load taken by a tie is equal to one-third of the wheel load is not acceptable. Consequently, the influence of potential differential elevation of the ties in an actual system should be assessed during the field appraisal program.

3. LOAD DISTRIBUTION TESTS

The set-up used for full scale laboratory testing of the open deck bridge system is shown in Fig.7. Nine ties were supported on two longitudinal girders. Simulated wheel loads were applied by means of a loading beam to the rails, which were attached to the ties using Pandrol fasteners. Load was applied to the loading beam by jacks reacting against another beam to which were attached tie rods anchored to the substructure. To ensure proper seating of the ties, subsequent to fastening the rail, shims were inserted between the ties and the tie-girder pads.

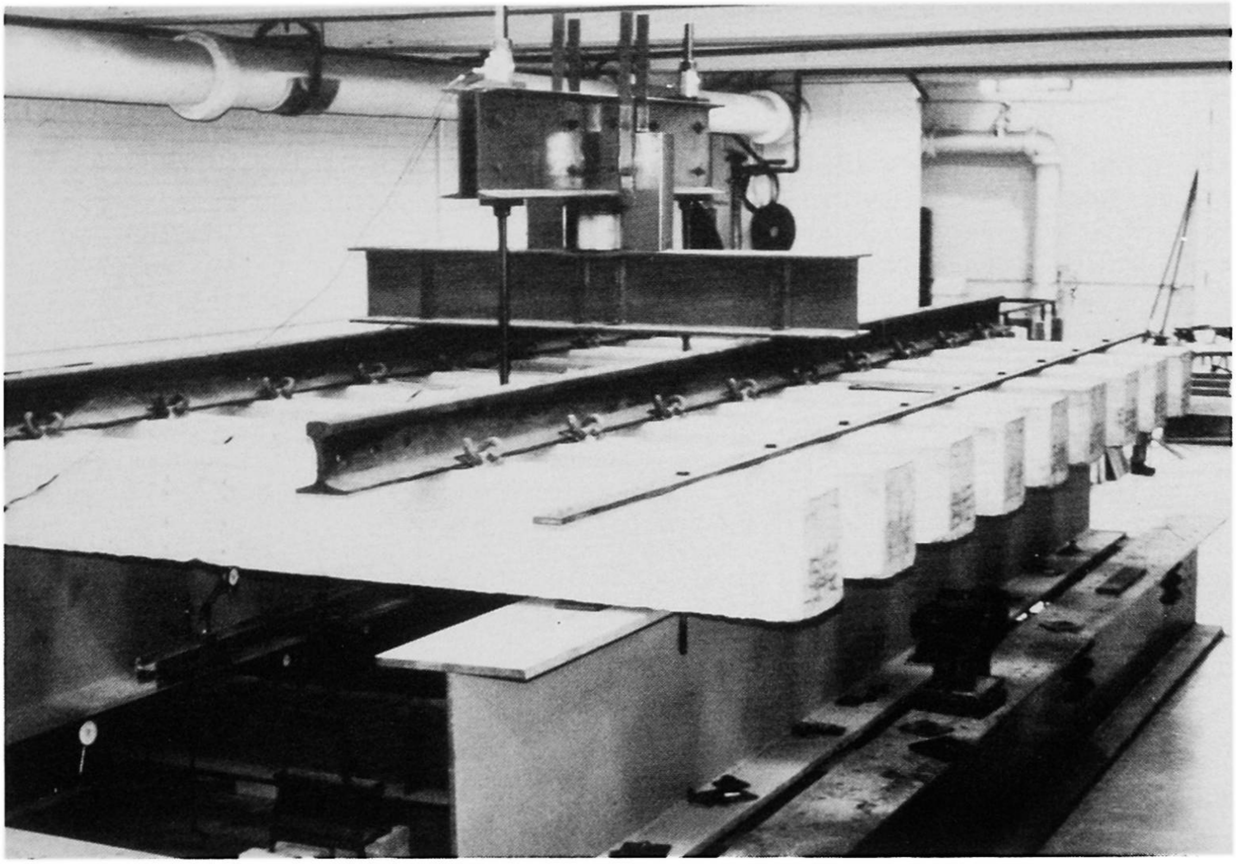


Fig.7. View of load distribution test

The load distribution among the ties was determined by measuring the flexural deformation of each of the nine ties by means of dial gauges. Each tie was calibrated prior to the load distribution tests by subjecting it to known simulated wheel loads and obtaining a load-deflection response curve.

The three significant parameters identified in the analytical study were varied in the load distribution tests. Results from a typical test are shown in Fig.8. Generally good agreement was obtained between the test data and the analytical predictions, thereby establishing the validity of the analytical model.

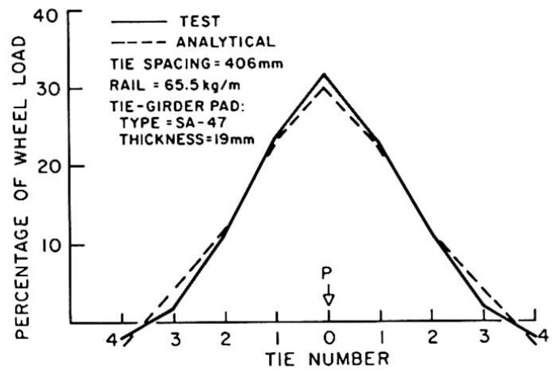


Fig.8. Comparison of test data with analytical prediction

Additional load distribution tests were conducted using the improper seating of the ties as obtained immediately after assembly of the various components. It was observed that, when the tie directly underneath the wheel loads was not seated properly, it did not carry the maximum load. Instead, the maximum load was carried by an adjacent tie. These tests showed an increase up to 60 percent in the maximum tie load in a system where all the ties were not properly seated. Further, these tests indicated a load distribution similar to that predicted by the analytical model in which gaps were considered to exist between the ties and the tie-girder pads at the relevant locations. The significant variation in load distribution suggests a strong need for a rigorous dimensional control and proper seating of all ties in an actual open deck bridge system.

4. FUTURE DEVELOPMENTS

Field testing of concrete ties in an open deck bridge system is currently being undertaken by the Canadian Railways. Load distribution, as well as the improper seating problem, are being studied in two bridges located on mainline track. Impact loads experienced by the ties are also being measured in order to establish realistic design loads for concrete bridge ties.

This open deck bridge system may be applicable to elevated guideways for rapid transit vehicles, where compatibility problems arise between the guideway beams and direct fixation continuous welded rail as a result of temperature variation. Use of ties on the girders effectively isolates the rail from the supporting structure and thus differential movement could be accommodated. This application requires further study.

REFERENCES

1. MIRZA, M.A., IGWEMEZIE, J.O. and IGWE, O.R., Load Distribution Characteristics of an Open Deck Bridge System with Precast Prestressed Ties, Experimental-Analytical Study. Final Report on Project P-8203, submitted to The Natural Science and Engineering Research Council, Ottawa, Canada, 1984, 208 pp.

2. CAMPBELL, T.I., KONG, W.L. and KWAN, P., Precast Prestressed Concrete Railway Bridge Ties. Final Report on Project P-8202, submitted to the Natural Science and Engineering Research Council, Ottawa, Canada, 1984, 176 pp.

3. BATHE, K.J., WILSON, E.L. and PETERSON, F.E., SAP IV - A Structural Analysis Program for Static and Dynamic Response of Linear Systems. Report No. EERC 73-11, University of California, Berkeley, 1974.

4. MANUAL FOR RAILWAY ENGINEERING, (Chapter 7), American Railway Engineering Association, Washington, D.C., USA, 1982.

Conclusions to Seminar III
Transit Guideway Structures

C.W. DOLAN
Consult. Eng.
ABAM Engineers, Inc.
Seattle, WA, USA

Two themes which made a common appearance in all of the papers discussing aerial transit structures were innovation and public concern. Innovation was the direct result of the special requirements of transit structures not being satisfied by conventional engineering solutions. The adjustable bearings on the TVE and the "creep couplers" of the German railroad structures are but two examples.

To meet the exacting tolerance requirements of their respective structures, both Mr. Gandil of France and Mr. Schambeck described methods of adjust beam post-tensioning and alignment throughout the structure life. Reinforcing this concern was Mr. Gandil's description of the monitoring program for the TGV train. Publication of the results of this long-term monitoring program should be encouraged to further advance our understanding of the performance of these structures.

Concern for social issues was a dominant theme of all the authors. Visual appearance and construction within the urban environment were principal concerns of both the ALRT and the Marseille transit systems. The concern for the public went beyond the visual appearance to the very root of structural design during the discussion of transit design criteria. Safety and reliability issues were also classified during the general discussion when the statement was made that transit systems should have a higher reliability than automotive bridges because the alternative routes do not generally exist.

All of the authors maintained an awareness of their respective construction procedures. The TVE magnetic levitation test track went the farthest, by considering special finishing equipment to fine tune the structure to precise tolerances. Mr. Prommersberger's description of coordination of rail joints and Mr. Croc's description of construction in historic portions of Marseille indicated both a technical awareness of structure needs and a realism of the constraints of the constuction industry.

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