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WORKSHOP 1

High Speed Rail Bridges

Ponts pour les trains à grande vitessse

Eisenbahnbrücken für hohe Geschwindigkeiten

Elevated Guideways of Systems for Tracked Transport

Voies de circulation à guidage imposé, élevées sur piliers

Aufgeständerte Fahrwege für spurgeführte Verkehrssysteme

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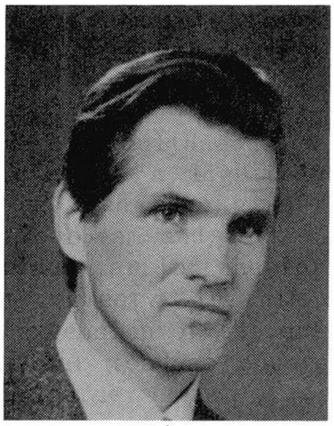

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SUMMARY

Elevated guideways are proving more and more to be the only possibility for building new sections for tracked transport systems. The requirements and their effect on the design and construction methods are explained with the help of several examples of elevated guideways for magnetically levitated transport systems and railway systems.

RESUME

La surélévation des voies de circulation est très souvent la seule possibilité pour réaliser un nouveau projet de transport à guidage imposé. Les exigences et leurs incidences sur l'étude et les méthodes de construction sont illustrées par quelques exemples de voies surélevées construites pour le système de transport magnétique et le chemin de fer.

ZUSAMMENFASSUNG

Die Aufständerung des Fahrweges ist immer häufiger die einzige Möglichkeit, eine neue Strecke für spurgeführte Verkehrssysteme zu bauen. Die Anforderungen und deren Auswirkung auf Entwurf und Bauverfahren werden an einigen Beispielen aufgeständerter Fahrwege für die Magnetbahn und die Eisenbahn erläutert.

1. INTRODUCTORY REMARK

More than 150 years ago the development of the railway as a large scale transportation system was begun, an important prerequisite for industrial progress. In those days the tracks were laid as far as possible on ground level or on embankments, and bridges were only built to overcome such obstacles as roads, rivers or deep valleys. Nowadays, however, it is often found that the only possible means of building a new transportation route is to elevate the whole line, i.e. to raise the track to a higher level. This insures unimpaired use of the terrain below and avoids the severing of the landscape by the traffic line.

In order to compete with the automobile and the airplane a tracked mass transportation system must operate at high speeds. For this reason, along with the further development of the wheel-on-rail system, the magnetic high-speed system with contact-free levitation technology (MagLev) was developed, which is currently being examined at test facilities under realistic conditions. (Fig. 1)

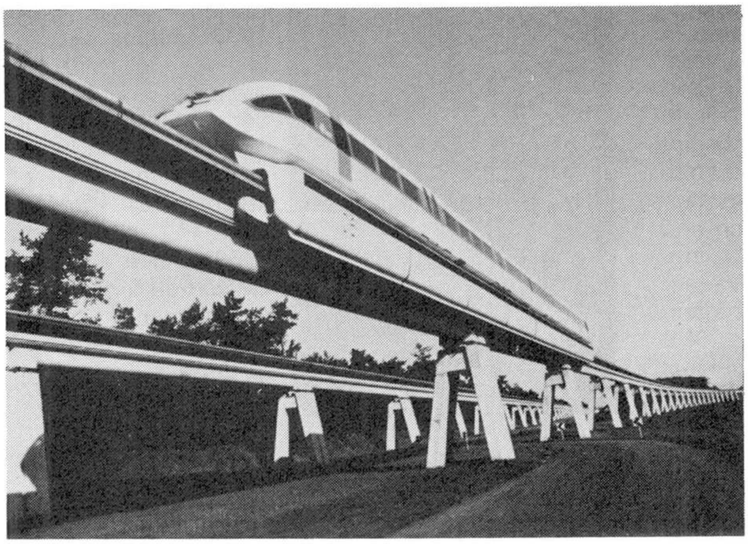

Fig. 1 Concrete guideway for the TRANSRAPID Test Facility Emsland, 1st construction phase with MagLev vehicle

2. GUIDEWAYS FOR HIGH-SPEED MAGLEV TRANSPORTATION SYSTEMS

2.1 System-specific requirements

Such transportation systems place specific demands on the guideway, which are determinative for its design as well as its construction methods:

* Structural system of the guideway in form of a girder, which is embraced by the vehicle and to which the functional components (in place of conventional rails) are fastened.
* A positional accuracy of the functional components unusually high for structures must be guaranteed.
* The structural system must possess a high rigidity and be insensitive to vibrations.

2.2 Elevated Guideway

2.2.1 General Remarks

The requirements stated above can be satisfied both economically and aesthetically by the use of freely spanned girders with a span of 25 to 35 m, which are mounted on piers of sufficient height to ensure the uninhibited use of the underlying terrain.

Because of the required rigidity and insensitivity to vibration of the structural systems prestressed concrete for the girders and reinforced concrete for the substructures as construction materials have proven advantageous.

Needed is a highly economical assembly procedure which satisfies strict tolerance requirements and a design whose longterm deformations are about zero and which allows unavoidable soil settlement to be quickly compensated.

This is achieved by the use of prefabricated girders, which ensures fast construction progress without regard to weather conditions, high accuracy and controlled quality of the materials.

Dyckerhoff & Widmann AG planned and, together with partners, built a guideway based on the above principles for the TRANSRAPID Test Facility Emsland (TVE) in northwestern Germany, which was erected in several construction phases between 1980 and 1987. This guideway, which is elevated along nearly its full length, has in general a clearance of 4.70 m above ground level and is single-track in compliance with the needs of the test facility. By the summer of 1990 the TRANSRAPID MagLev vehicles had covered more than 100,000 km on the test route achieving speeds up to 435 km/h.

The guideway, consisting of a straight-away and two turning loops, has a total length of ca 31 km. Besides the concrete guideway, a description of which follows, the test course also contains sections with girders of steel, [1], [2].

2.2.2 Design of the concrete guideway for the TVE

In order to fulfill the extremely strict requirements for positional accuracy of the functional components, the design is based on a strict separation of the two installation phases erection of the supporting structures and mounting of the functional components (Fig. 2). This allows greater tolerances in the supporting structures, which can be compensated for later by adjustments made when the functional components are installed. Thus initial deformations in the green concrete of the girders have no influence on the positional accuracy of the functional components.

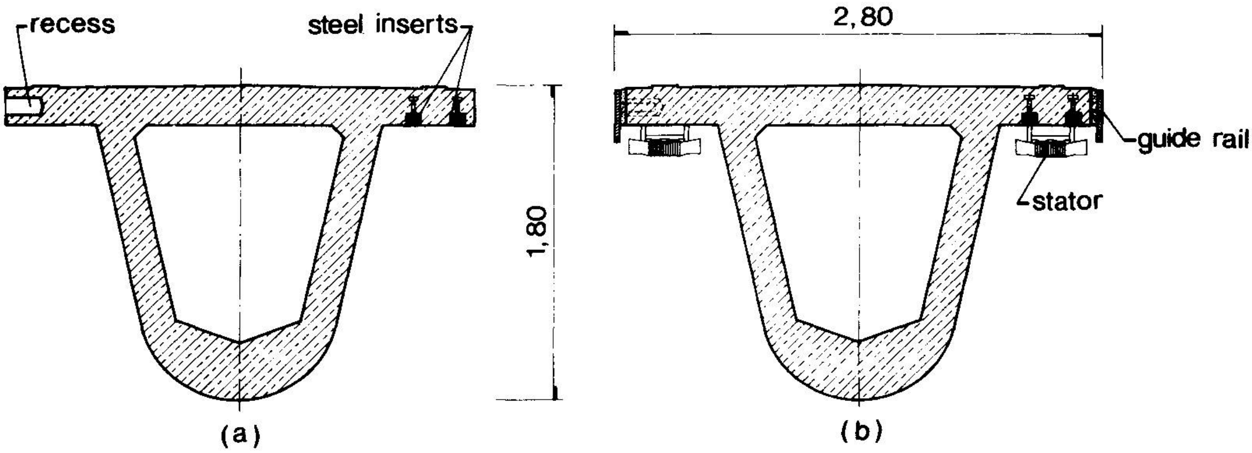

Fig. 2 Girder cross section TVE, 2nd construction phase
(a) before, (b) after installation of the functional components

The hollow-box girders of the guideway are of prestressed concrete and have a cantilevering deck slab. As single-span girders they have a span of ca 25 m with a structural height of 1.80 m. There are also spans of ca 31 m at a structural height of 2.40 m as well as a few other spans.

Essential for a long-term maintainance of the original high positional accuracy of the funtional components is the non-distortional prestressing of the track girders, which ensures that practically no plastic deflection occurs. This is achieved by the radial forces of the curved prestressing tendons counteracting the dead load and providing average compensation for this load.

Because of the strong eccentric loads carried by them, especially in curves with max. 12° cant, the piers are A-shaped, as seen in Fig. 3, with two slender, slanting slabs set in the connecting slab of the piles and running together at the pier head, upon which the structural bearings of the track girders, laterally spread, are positioned.

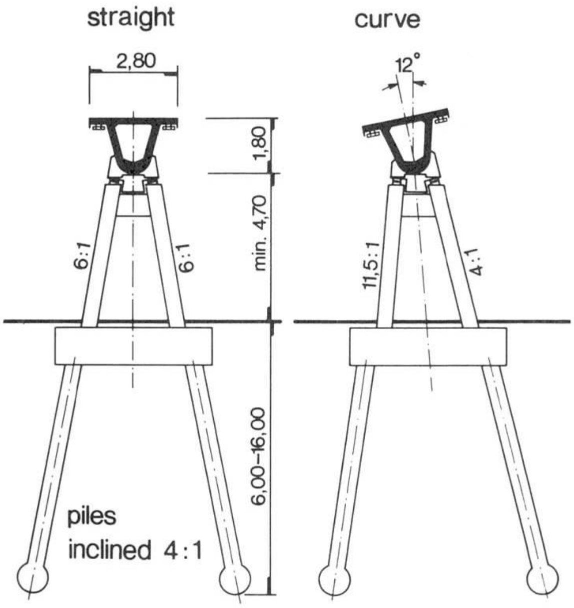

Fig. 3 Cross section of the TVE guideway, span 25 m

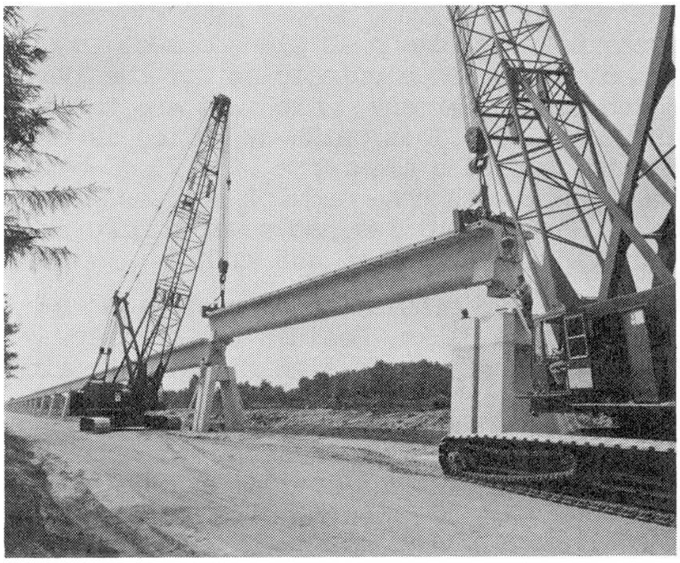

Fig. 4 Installation of the track girder TVE, 1st construction phase

2.2.3 Construction procedure for the TVE concrete guideway

- The foundations and piers were erected in concrete cast in-situ by means of a wandering construction site, which progressed at an average of three piers per day, ca 75 - 90 m.
- The track girders, altogether about 800 in number, were produced as prefabricated prestressed concrete elements in a field factory at the rate of one girder per day in each of two, and at times three, moulds operating simultaneously. The fast rate of production was made possible by prefabricating the mesh of reinforcing steel and heating the green concrete, [1].
- The installation of the track girders at the construction site was accomplished by use of mobile cranes with caterpillar drive (Fig. 4).
- With the help of hydraulic jacks the girders were finely positioned and then set on the bearings by filling in with mortar.

2.3 MagLev guideways for public service

The cost of the guideway and its compatibility with the environment are decisive factors for the realization of new transportation routes. For this reason there was an early activity in Germany starting route studies for MagLev systems and setting up a basic design for typical guideway constructions.

2.3.1 The alignment of MagLev lines

Because of the derailment-proof clasp of the track combined with great cant a MagLev line can be aligned with comparatively small radii of curvature and because of the great climbing ability of the long-stator propulsion it can be aligned with steep gradients. As a result it can be favorably adapted to the terrain through which it passes.

The limiting alignment elements (for instance for speeds of 400 km/h and 500 km/h respectively) are

	400 km/h	500 km/h
Maximum cant	12°	12°
Minimum radius of curvature (plan)	4 180 m	6 530 m
Minimum vertical radius for summits	24 700 m	38 580 m
Minimum vertical radius for sags	12 350 m	19 290 m
Maximum gradient	10 % (1:10)	10 % (1:10)

2.3.2 Typical guideway constructions

For public service lines double-track guideways are generally needed which can either be elevated or constructed on ground level or, depending upon the terrain, be continued over bridges or through tunnels. For example Fig. 5 shows an elevated double-track guideway built of prestressed concrete. It consists of two separate girders set side by side on a common substructure. The girders can be designed to span 35 m at a construction height of 2 m. The substructures consist of slender double columns adaptable to differing clearances. The girders are prefabricated and then installed by means of mounting devices. In difficult terrain special devices can be moved along on the already available supports.

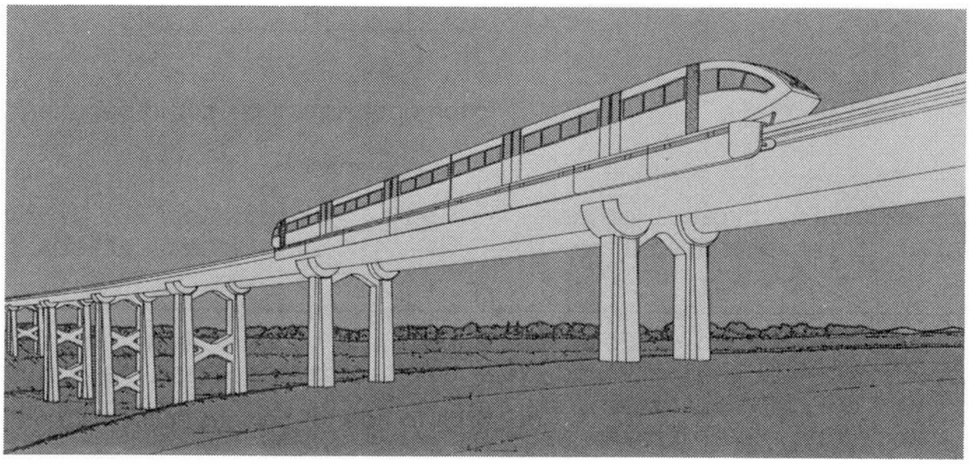

Fig. 5 Elevated double-track guideway, drawing in perspective

3. ELEVATED RAILWAY LINES

As already mentioned a lack of necessary space for the construction of a new railway line can also lead to a decision to elevate the track. A good example of this is the urban rail system Metro Medellin now under construction in Columbia, which will connect the suburban areas with the city center, [3]. Since an underground line would have been too expensive, the only other possibility to realize an intersection-free route over a length of about 11 km within the inner city area was to raise it 7 to 19 m above street level. The rest of the Metro Medellin network, which has a total length of about 30 km and which was planned and built by a Spanish-German consortium, is located on the outskirts of the city and runs on ground level.

Within the consortium Dyckerhoff & Widmann AG was responsible for the design work for the 11 km of elevated track, including 13 elevated stations. The superstructure for the double-track elevated line with ballast bed, as shown in Fig. 6, consists of prefabricated post-tensioned concrete girders, each generally 30 m in length with a weight of 2300 kN, supported by a number of center piers of concrete cast in-situ, which usually follow the median strip on main thoroughfares. Under each track is a seperate girder.

To allow for clearance and for architectural reasons the girders are shaped at both ends to rest on hidden hammerheads on the piers. Thus the elevated railway appears as a light and well balanced structure (Fig. 7).

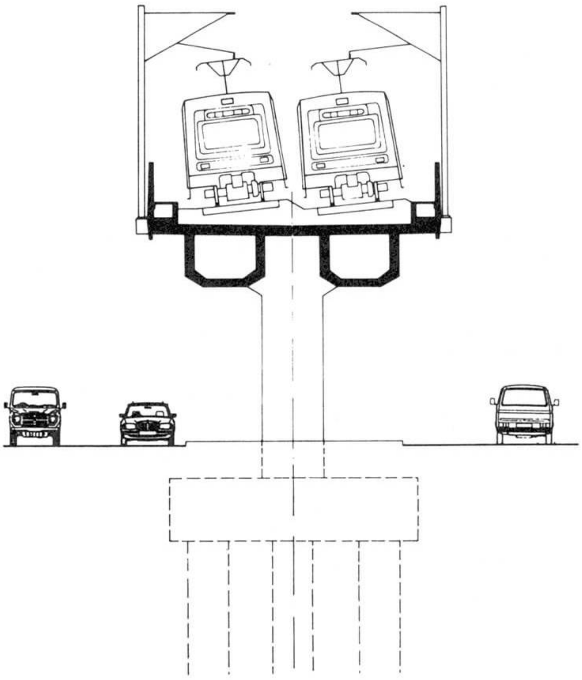

Fig. 6 Metro Medellin, typical cross section of the elevated line

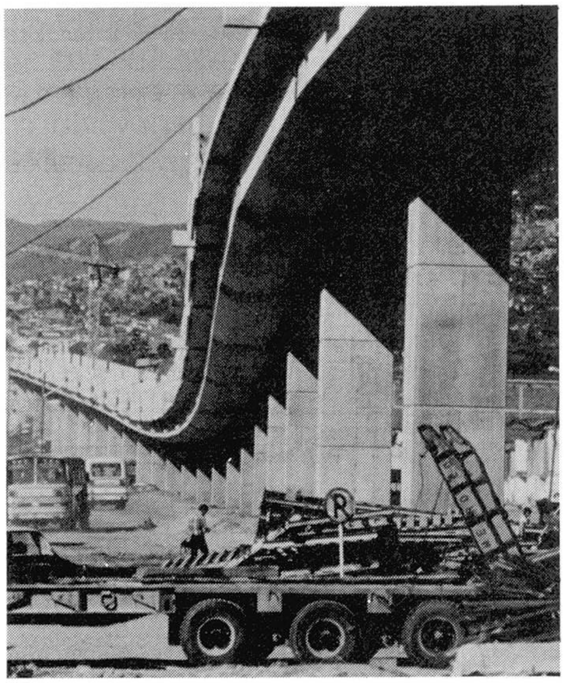

Fig. 7 Metro Medellin, elevated line under construction

The construction procedure had to be planned in such a way as to hinder the traffic as little as possible. For this reason the girders were prefabricated in field factories outside the city and then mounted by a special launching truss, which can travel along the hammerheads of the center piers leaving the finished superstructure behind. The girders were delivered to the launching truss by trucks designed for this purpose, which can travel along the already finished superstructure. The maximum transport distance was about 4 km.

4. CONCLUSION

With prestressed concrete elevated lines for tracked high-speed transportation systems can be constructed in an economical as well as aesthetically satisfying manner. High standards are required for accuracy of execution and long-term maintainance of the specified form. When design and construction are in the hands of an experienced company with high technical standards, the prerequisites for an optimal realization of this type of structures are assured.

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Ouvrages d'art pour un réseau ferré à très grande vitesse

Kunstbauten für Hochgeschwindigkeitsbahnen

Bridge Structures for High Speed Rail Systems

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RESUME

Le développement du réseau ferroviaire à grande vitesse se poursuit à un rythme soutenu en France. L'évolution des conditions d'exploitation et l'influence croissante des contraintes d'environnement rendent la part des ouvrages d'art toujours plus importante dans les projets de lignes nouvelles. La conception de ces ouvrages évolue également en fonction des exigences nouvelles, des progrès technologiques, d'expériences et des données économiques.

ZUSAMMENFASSUNG

Die Entwicklung der Schnellbahnen in Frankreich schreitet rasch voran. Die Kunstbauten werden aufgrund der einschränkenden Umweltanforderungen zu immer wichtigeren Projektteilen. Der Entwurf dieser Bauten entwickelt sich mit den neuen Anforderungen, dem technischen Fortschritt, den Erfahrungen und den wirtschaftlichen Bedingungen.

SUMMARY

The development of high speed railways in France is progressing rapidly. Operational and environmental requirements have an influence on the bridge structures which are becoming increasingly important parts of the project. The design of these structures is evolving to match new requirements, technical progress, experience and economic conditions.

1. LE DEVELOPPEMENT DE LA TRES GRANDE VITESSE EN FRANCE

Dès 1968, la S.N.C.F. engageait les premières études de faisabilité d'une ligne à grande vitesse desservant le Sud-Est de la France. La mise en service commerciale de la Ligne Nouvelle à Grande Vitesse "T.G.V. SUD-EST", longue de 400 km entre Paris et Lyon, était achevée en septembre 1983. Cette ligne est exploitée à 270 km/h (prochainement 300 km/h).

Le succès et la réussite de la mise en service partielle en 1981 avaient permis à la S.N.C.F., dès 1982, d'être autorisée à réaliser une deuxième ligne, le "T.G.V. Atlantique", qui dessert l'Ouest et le Sud-Ouest de la France depuis septembre 1990 à 300 km/h.

Plus récemment, et consécutivement à la décision de construire le lien fixe Transmanche "Tunnel sous la Manche", le gouvernement français, favorable à la création d'un véritable réseau Très Grande Vitesse, a demandé à la S.N.C.F. d'engager la construction de trois nouveaux projets représentant 530 km de lignes :
- le T.G.V. Nord (mise en service prévue en 1993)
- l'interconnexion des lignes T.G.V. en région parisienne (mise en service prévue en 1995)
- le prolongement jusqu'à Valence du T.G.V. Sud-Est dit "T.G.V. Rhône-Alpes" (mise en service prévue en 1994).

C'est enfin en janvier 1989 que le gouvernement a chargé la S.N.C.F. de présenter un projet de Schéma Directeur des Liaisons Ferroviaires à Grande Vitesse "en vue notamment d'assurer la cohérence nécessaire entre les perspectives de développement du réseau français et les projets correspondants des autres pays européens".

3132 km de lignes nouvelles supplémentaires ont ainsi été proposés, et la S.N.C.F. a été autorisée en Janvier 1991 à engager les études en vue de la réalisation du T.G.V. Provence - Côte d'Azur en prolongement du T.G.V. Rhône-Alpes.

La S.N.C.F. a donc pu se forger une réelle expérience dans le domaine de la grande et de la très grande vitesse, et notamment en ce qui concerne les ouvrages d'art. Les solutions techniques retenues doivent cependant s'adapter à l'évolution du contexte et à l'augmentation des contraintes.

2. L'EVOLUTION DES CONDITIONS D'EXPLOITATION

2.1 L'augmentation des vitesses

La ligne Paris - Lyon, exploitée à 260 km/h à sa mise en service, puis à 270 km/h, permet par sa géométrie une vitesse de 300 km/h (qui sera d'ailleurs bientôt pratiquée).

Ces vitesses étaient jugées à l'époque comme représentant l'optimum économique entre les coûts d'exploitation et les recettes de l'augmentation de trafic. Les progrès technologiques aidant, les coûts ont baissé, pendant que parallèlement l'induction de trafic se révélait plus forte que prévue.

L'optimum s'est donc déplacé.

C'est pourquoi la ligne Atlantique vient d'être mise en service à 300 km/h, et le T.G.V. Nord le sera sans doute à 320 ; les tracés de ces lignes permettent toutefois 350 km/h. Les études de faisabilité réalisées dans le cadre du projet de Schéma Directeur français prennent en compte, quand la géographie et l'environnement le permettent, des vitesses de l'ordre de 400 km/h. Le récent record du monde (515,3 km/h) réalisé sur la ligne T.G.V. Atlantique a montré que des vitesses largement supérieures à 300 km/h pouvaient être pratiquées en toute sécurité.

L'augmentation des vitesses a une incidence sur les tracés (les rayons nominaux étant de 4000 mètres à 270 km/h, 6000 mètres à 350 km/h, 8000 mètres environ à 400 km/h), sur les profils en long (augmentation des rayons de raccordement) et par là même sur le nombre et les dimensions des ouvrages d'art, les lignes épousant plus difficilement le terrain naturel.

Les principaux effets de la vitesse sont les sollicitations dynamiques pour les ponts-rails et les viaducs (vibrations) et les phénomènes aérodynamiques dans les tunnels.

2.2 La mixité du trafic voyageurs/marchandises

L'étendue du territoire français et la faible densité des grandes agglomérations ont conduit à construire et à prévoir des lignes nouvelles pour la plupart spécialisées au trafic voyageurs.

Ce choix a favorisé la souplesse de l'exploitation et a été un facteur important du faible coût de maintenance que nous avons constaté en 10 ans sur le T.G.V. Sud-Est.

Néanmoins certains contextes régionaux sont tels que l'intérêt économique de tronçons mixtes voyageurs-marchandises est démontré. Ainsi, le contournement de la ville de Tours par la ligne Atlantique, déjà en service, est mixte ; seront de même le tronçon de ligne nouvelle entre Perpignan et la frontière espagnole et une partie de la ligne nouvelle Lyon - Turin prévue au Schéma Directeur, par un tunnel de base sous le Mont Cenis.

L'emprunt par des trains de marchandises des lignes à grande vitesse nécessite de limiter les rampes (15 ‰ au lieu de 25, voire 35 ‰) et d'augmenter les rayons des courbes (pour rendre compatibles aux deux types de trafic le dévers de la voie).

L'inscription plus difficile dans le contexte géographique qui en résulte influe, comme la vitesse, sur le nombre et la portée des ouvrages de franchissement.

3. LA SENSIBILITE ACCRUE AUX PROBLEMES D'ENVIRONNEMENT

La construction de la ligne Paris - Lyon n'a pas posé de problème aigu d'environnement, puisque sur ses 400 km approximatifs, elle ne rencontrait aucune agglomération importante, certaines zones traversées étant même particulièrement peu peuplées.

Au fur et à mesure des projets, T.G.V. Atlantique puis T.G.V. Nord et maintenant T.G.V. Méditerranée, les lignes ont rencontré des zones de plus en plus complexes sur le plan de l'urbanisation, de la qualité de l'agriculture, de la richesse de certaines zones naturelles.

La densification des réseaux de communication a multiplié les franchissements d'infrastructures.

Parallèlement à cette complexité technique, des textes concernant la protection des milieux naturels et de l'environnement ont été publiés (il n'en existait pas en France à l'époque de la décision de construire la ligne Paris - Lyon), obligeant les maîtres d'ouvrage - en l'occurrence la S.N.C.F. - à réaliser un certain nombre d'études et de mesures particulières.

Pendant la même période, la sensibilité des populations à ces problèmes n'a fait que croître.

En matière d'environnement, la S.N.C.F. n'a cessé de perfectionner ses méthodes d'étude, ses actions, au fur et à mesure de ses projets. Bien sûr, ceci s'est traduit par un certain renchérissement des coûts, puisque nous estimons entre 12 et 15 % du coût d'un projet les mesures de protection de l'environnement. En particulier, il a fallu s'orienter vers une certaine densification des ouvrages d'art ; alors qu'aucun tunnel n'avait été réalisé sur la ligne Paris - Lyon, il y en a sur la ligne Atlantique, en particulier l'ouvrage dit "coulée verte" qui permet aux trains d'accéder au coeur de la capitale.

4. L'AUGMENTATION DE LA PART "OUVRAGES D'ART" DANS LES PROJETS

Comme nous l'avons vu, contrairement au T.G.V. Sud-Est où le nombre des grands ouvrages d'art avait pu être très limité, les projets en cours comportent de nombreux viaducs, sauts de mouton, tunnels et tranchées couvertes, dont les dimensions vont croissantes : augmentation des portées pour les viaducs, augmentation des sections des tunnels et tranchées couvertes du fait de l'élévation de la vitesse des trains.

Les chiffres suivants, indiquant les coûts d'Ouvrages d'Art, par kilomètre de ligne, aux conditions économiques identiques à celle de 1989, confortent ce propos :

T.G.V. Sud-Est (1983)	: 3,1 Millions de Francs/Km
T.G.V. Atlantique (1990)	: 12,1 Millions de Francs/Km
T.G.V. Nord (1993)	: 11,0 Millions de Francs/Km
T.G.V. Rhône-Alpes (1994)	: 19,6 Millions de Francs/Km
T.G.V. Interconnexion (1995)	: 27,4 Millions de Francs/Km

La longueur des plus grandes viaducs suit la même progression : de 420 mètres sur le T.G.V. Sud-Est à 1800 mètres sur les projets en cours de réalisation.

Il est tenu compte de l'expérience acquise sur les lignes T.G.V. exploitées pour la conception des ouvrages des lignes T.G.V. en cours de réalisation, ceci en matière de caractéristiques fonctionnelles des ouvrages, choix de type de structures, dispositions constructives particulières.

En effet, avec le T.G.V. Paris-Sud-Est et le T.G.V. Atlantique, la S.N.C.F. dispose d'un patrimoine existant de 700 ouvrages en exploitation, dont 300 sont des ponts-rails de types et de conception variés : viaducs de type caisson en béton précontraint, tabliers dalles ou bipoutres isostatiques en béton armé, tabliers à poutrelles enrobées, ponts-cadres, tabliers à 1 voie, à 2 voies, etc...

Un grand nombre ont été parcourus à des vitesses de 400 km/h, certains à plus de 500 km/h.

Nous allons maintenant aborder plus précisément les ponts et grands viaducs.

5. LA CONCEPTION DES PONTS-RAILS ET VIADUCS SUR LES LIGNES A GRANDE VITESSE FRANCAISES

5.1 Le T.G.V. Sud-Est

Les ouvrages ont été conçus pour des vitesses d'exploitation de 300 km/h, conditionnant les gabarits (entre-axe des voies porté à 4,20 mètres) et la flexibilité des tabliers. La pérennité maximale a été recherchée en vue de réduire les interventions d'entretien.

La très grosse majorité des ouvrages est donc de type courant pour les ponts-rails : dalles béton armé à 1 travée, tabliers à poutrelles enrobées, portiques et cadres en béton armé...

Tous les tabliers de ces ponts sont ballastés : le ballast améliore l'amortissement des vibrations, assure un bon comportement dynamique de la voie aux grandes vitesses, assure un meilleur confort sonore et permet de limiter aux grands ouvrages la mise en place d'appareils de dilatation de la voie. Ces dispositions facilitent grandement la maintenance de la voie.

Les viaducs, dont le plus grand mesure 420 mètres de longueur, ont été réalisés avec des tabliers en béton précontraint constitués généralement d'un caisson à deux âmes portant les deux voies. Cette solution, bénéficiant de l'expérience acquise à l'occasion des premiers ouvrages construits sur les lignes de desserte des villes nouvelles en région parisienne, a été adaptée en fonction des exigences de la grande vitesse ferroviaire. Elle s'est avérée une bonne réponse aux problèmes de bruit, de vibration (forte inertie du tablier) et de raideur.

5.2 Le T.G.V. Atlantique

Le faible temps entre la mise en service du T.G.V. Sud-Est et le début de la réalisation du T.G.V. Atlantique n'a pas permis une expérience susceptible de remettre en cause les dispositions constructives retenues. C'est pourquoi l'évolution est peu sensible dans la conception des ponts-rails et grands viaducs dont le plus long atteint 840 mètres.

En revanche, pour ces deux premières lignes à grande vitesse françaises, un bilan significatif a pu être établi s'appuyant sur les constats faits à l'occasion des visites annuelles, des inspections détaillées, des visites spéciales programmées. Ce constat est enrichi par ailleurs des nombreux résultats acquis sur les viaducs grâce à une instrumentation particulière de ces ouvrages, à l'aide de cordes vibrantes disposées dans les tabliers et de pesons installés sous chaque appareil d'appui. Un large recours a été fait par ailleurs à des mesures à l'aide d'accéléromètres afin de bien analyser le comportement vibratoire des tabliers au passage des circulations et l'évolution dans le temps.

Les constatations faites montrent un comportement global très satisfaisant des ouvrages, de leurs équipements et des superstructures. Seuls faits significatifs à signaler, le faible amortissement des tabliers isostatiques à poutrelles enrobées posés sur appuis élastomères, quelques phénomènes d'attrition du ballast sous traverses, sur hourdis de tablier, dans certaines circonstances.

Ainsi, pour les lignes actuellement en cours de travaux, la S.N.C.F. a pu bénéficier de 5 années d'expérience, tout en profitant de l'évolution de la technologie et des données économiques.

5.3 Les T.G.V. en cours de réalisation

Il s'agit du T.G.V. Nord, du T.G.V. Rhône-Alpes et de l'Interconnexion en région parisienne.

Les ouvrages sont conçus, d'une manière générale, pour des vitesses d'exploitation de 350 km/h.

De nouveaux types de ponts sont apparus, notamment les ouvrages mixtes acier-béton à tablier bipoutre. Ceux-ci, qui représentent environ 50 % des viaducs pour les 3 lignes concernées, sont bien adaptés à la pose de voie ballastée, et, par leur structure simple, sont peu sensibles à la fatigue et à la corrosion.

Ils ont bénéficié de progrès caractérisés :
- par une amélioration des caractéristiques des aciers et des épaisseurs disponibles
- par une rationalisation des procédés de construction en usine et sur chantier (soudage - contrôle du soudage)
- par l'amélioration des traitements anti-corrosion
- par l'amélioration des méthodes de calcul automatique, tant en statique qu'en dynamique.

Ces ouvrages, dont les coûts sont devenus compétitifs, utilisent les deux matériaux au mieux de leurs possibilités mécaniques, et s'avèrent d'un entretien facile et peu onéreux, les opérations d'entretien se faisant essentiellement sans répercussion sur les circulations.

Actuellement, nous maîtrisons le comportement dynamique des ouvrages sur lignes à grandes vitesses pour des portées avoisinant 100 mètres ; les plus grands viaducs projetés approchent les 2000 mètres de longueur.

En ce qui concerne les ouvrages plus modestes (pont-rails courants par exemple), on recourt aux tabliers hyperstatiques pour bien maîtriser leur comportement dynamique.

6. LES OUVRAGES D'ART DU RESEAU T.G.V. DU FUTUR

Comme nous l'avons expliqué, les futures lignes à très grande vitesse seront riches en ouvrages d'art dont les dimensions ne cesseront de croître.

Ainsi, nous réfléchissons à la faisabilité :
- de viaducs de portées supérieures à 150 mètres, comportant des tabliers situés à des hauteurs supérieures à 100 mètres,
- de tunnels de plus 50 km de long,
- de gares souterraines circulées à grande vitesse.

Dans le domaine des Ouvrages d'Art, comme de la grande vitesse en général, la S.N.C.F. entend jouer non seulement la carte de l'expérience, mais également celle du progrès technologique continu.

Comportement dynamique des ponts-rails sous lignes à grande vitesse

Dynamisches Verhalten von Eisenbahnbrücken auf Hochgeschwindigkeitsstrecken

Dynamic Behaviour of Railway Bridges for High Speed Lines

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RESUME

Le comportement dynamique d'un ouvrage d'art à grande vitesse pour une circulation ferroviaire doit faire l'objet d'études particulières. Après avoir analysé l'incidence de la vitesse sur ce comportement et les conséquences qui en résultent en matière de sécurité et de confort, cet article décrit les justifications spécifiques de cacul qu'entraîne la grande vitesse. Enfin, sont données, en ce qui concerne les ouvrages mixtes, les dispositions constructives retenues à la SNCF pour répondre à ces exigences.

ZUSAMMENFASSUNG

Das dynamische Verhalten von Kunstbauten als Bestandteil von Hochgeschwindigkeits-Eisenbahnstrecken erfordert besondere Studien. Nach einer Analyse der Auswirkung der Geschwindigkeit auf das Tragwerksverhalten in Bezug auf Sicherheit und Komfort legt der Aufsatz die spezifischen Berechnungsgrundlagen dar. Schliesslich werden konstruktive Detailvorschriften für Verbundbrücken beschrieben.

SUMMARY

The dynamic behaviour of bridge structures for high speed railway lines requires specialized studies. After an analysis of the effects of speed on the structural behaviour related to safety and passenger comfort the paper presents the specific computational fundamentals. Finally, the detail requirements for composite structures for the French Railways are described.

1. INTRODUCTION

Situé sur une voie de communication pour en assurer la continuité, un ouvrage d'art doit garantir aux circulations l'empruntant le même degré de sécurité et de confort que celui existant par ailleurs. Ceci est particulièrement vrai dans le domaine ferroviaire où la sécurité régit les actions et les pensées de tout cheminot. Ceci est encore plus vrai dans le domaine de la grande vitesse car, d'une part les effets dynamiques y accentuent tous les phénomènes pouvant avoir une incidence sur la sécurité, d'autre part s'agissant d'un moyen de transport de haute technologie, il serait inconcevable que le confort n'y soit pas optimum.

La S.N.C.F. dispose déjà d'un patrimoine important d'ouvrages sous lignes T.G.V. exploitées, certains d'entre eux ayant même été parcourus à des vitesses supérieures à 500 km/h. Tous ces ouvrages cependant sont soit en béton armé, soit en béton précontraint, soit à poutrelles enrobées. Les constatations faites sur ces premières familles d'ouvrages, comme les progrès réalisés ces dernières années par la construction métallique, ont permis à la S.N.C.F. de projeter plusieurs viaducs métalliques ou mixtes sur les lignes à grande vitesse en cours de construction (voir fig. 1). Ces types d'ouvrages avaient été absents jusqu'alors sur les lignes T.G.V. en exploitation, d'une part en raison d'une faible compétitivité au plan économique, d'autre part du fait que les études préalables permettant de s'assurer de leur bon comportement dynamique à grande vitesse n'étaient pas assez avancées.

2. INCIDENCE DE LA GRANDE VITESSE SUR LE COMPORTEMENT DES OUVRAGES

A des vitesses inférieures à 200 km/h, il n'existe que peu de phénomènes vibratoires entretenus, tout au moins pour l'ossature principale d'un ouvrage.

A grande vitesse, la fréquence excitatrice augmente par l'effet propre de la vitesse. Cet effet est lui-même fortement amplifié par l'utilisation de convois long (rame double) composé de matériels à écartement de bogies très régulier ; de ce fait, pendant le passage de la totalité de la rame n'apparaissent que peu de fréquences d'excitation secondaires susceptibles d'atténuer l'effet de l'excitation principale.

On peut se situer alors dans des plages de résonance où des amplifications dynamiques importantes peuvent se produire, et ce d'autant plus que la fréquence naturelle du pont non chargé est faible. Plus légers que leurs homologues en béton précontraint, les viaducs à ossature métallique peuvent y être particulièrement sensibles si aucune analyse de leur comportement dynamique n'est réalisée au stade des études préliminaires et si aucune disposition constructive spécifique n'est alors arrêtée. Les vérifications traditionnelles de résistance comme de déformations statiques ne sont plus suffisantes, il convient de procéder à une analyse du comportement vibratoire des tabliers.

3. SECURITE ET CONFORT

Des oscillations excessives des tabliers peuvent mettre en cause la sécurité des circulations ou tout simplement le confort des voyageurs.

3.1 La sécurité des circulations

La sécurité des circulations repose principalement sur la parfaite maîtrise en toutes circonstances :
- de la stabilité de la voie,
- d'une bonne qualité du contact rail-roue

3.1.1 La stabilité de la voie

En France, la pose de voie ballastée a été retenue systématiquement jusqu'à présent sur les lignes à grande vitesse pour les facilités d'entretien mécanisé et les possibilités de réglage qu'elle autorise. La voie est équipée en outre de longs rails soudés, ce qui permet d'améliorer le confort et de réduire les nuisances. Selon les longueurs dilatables des tabliers, selon leur condition d'appui, la voie comporte ou non également des appareils de dilatation de voie aux extrémités mobiles des tabliers.

En pleine voie, pour des conditions normales de pose, de réglage et d'entretien, les seuls paramètres influant de façon significative sur les efforts dans les rails sont la température et les efforts de freinage et démarrage. Les efforts longitudinaux et transversaux développés dans la voie en raison de ces deux effets sont repris par le ballast en butée latérale ou par le frottement des traverses.

Au franchissement d'un ouvrage d'art, l'intensité et la distribution de ces efforts sont considérablement perturbés du fait que la voie repose sur une structure dilatable et déformable contrairement à ce qui se passe en pleine voie où elle repose sur une plate-forme inerte thermiquement et quasi indéformable au regard des déformations des ouvrages.

Le maintien de la continuité de la voie à chaque extrémité des ouvrages est à l'origine dès lors de tout un champ d'efforts d'origines diverses dans la voie et dans l'ouvrage et qui peuvent être préjudiciables au bon comportement de l'une comme de l'autre.

De nombreuses actions ou effets sont à l'origine de ces efforts dits d'interaction "voie-ouvrages d'art". Il s'agit principalement :

a) des effets thermiques propres aux rails et aux tabliers,

b) des effets de freinage et de démarrage,

c) des déformations et oscillations de l'ouvrage induites par le passage des circulations.

Les effets a) et b) ne font pas l'objet du présent exposé ; il suffit de savoir cependant que la dilatation d'un tablier par forte température induit des efforts de compression très importants dans la voie côté extrémité mobile des tabliers, qu'en cas de freinage de circulation sur un tablier des efforts supplémentaires de compression très importants dans la voie viennent s'ajouter aux effets précédents.

Ces efforts de compression peuvent encore être aggravés en raison des déformations et des oscillations visées en c).

Au passage d'une circulation sur un tablier, en effet, ce dernier est le siège de déformations, d'oscillations, provoquant à l'extrémité mobile des tabliers des déplacements longitudinaux et des rotations des sections d'extrémité qui génèrent au niveau des rails des efforts. Ces efforts pour certains cas de chargement peuvent également être des efforts de compression qui viennent s'ajouter aux efforts dus aux effets thermiques (voir fig. 2) et aux effets de freinage.

Devant ces divers efforts de compression, le risque encouru est un flambage latéral de la voie. Pour s'y opposer, il faut disposer d'une excellente butée latérale offerte par le ballast et donc être assuré de la bonne cohésion de ce dernier en toutes circonstances.

Or, cette cohésion peut se trouver considérablement réduite :
- le long du tablier par les accélérations induites dans le ballast du fait des oscillations verticales du tablier,
- en extrémité du tablier par les petits déplacements longitudinaux oscillatoires résultant, au niveau du ballast, des déplacements d'ensembles longitudinaux du tablier et des rotations des sections d'extrémité induits par le passage des circulations.

3.1.2. Le contact rail-roue

La qualité du contact rail-roue sera assurée si la géométrie de l'ouvrage déformée est à tout instant compatible avec celle du convoi et si les accélérations verticales induites par les oscillations du tablier ne déchargent pas les essieux de façon excessive.

Pour assurer cette bonne qualité du contact rail-roue, trois limitations sont imposées au tablier en cours de déformation concernant respectivement les gauches pris par la voie en tout point du tablier, les déviations angulaires au droit des appuis principalement en extrémité de tablier, enfin les accélérations verticales induites dans les tabliers.

3.2. Le confort

L'analyse de la notion de confort, quant à elle, peut être plus rapidement faite : le confort des voyageurs est essentiellement affecté par le mouvement des voitures. Si les limites de gauche de voie résultant des considérations de sécurité sont suffisantes pour assurer un bon comportement transversal des voitures, il y a lieu, par une limitation de l'accélération verticale de la caisse, de s'assurer de leur bon comportement vertical.

Tout ce qui précède montre l'absolue nécessité de procéder à une analyse fine du comportement vibratoire des tabliers pour s'assurer qu'en toutes circonstances (y compris sous les amplifications résultant de phénomènes de résonance éventuels) les limitations prévues pour satisfaire aux problèmes de stabilité de la voie, de contact rail-roue et de confort seront respectées.

4. JUSTIFICATIONS PAR LE CALCUL

4.1 Considérations générales

Pour les ouvrages ordinaires et les conditions d'exploitation classiques, il était d'usage de régler les problèmes de sécurité et de confort par une seule obligation : ne pas dépasser telle valeur de flèche sous convoi de référence. Avec les grandes vitesses, il convient d'aller plus loin en imposant, ainsi qu'on vient de l'exposer, des limitations supplémentaires en matière d'accélération, de gauche et de déviation angulaire au droit des appuis d'ouvrages.

La réponse dynamique d'un tablier est fonction d'un certain nombre de paramètres très importants, notamment inertie, masse, amortissement dont la quantification est souvent malaisée :
- l'inertie d'un tablier évolue dans le temps du fait de son vieillissement (multiplication et propagation des fissures dans le béton par exemple),
- sa masse peut augmenter ou diminuer selon que les opérations d'entretien mécanisé de la voie conduisent à augmenter ou à réduire les épaisseurs de ballast.

On est ainsi conduit :

- pour la détermination des fréquences propres , (de la forme $f_i = k_i\sqrt{\frac{I}{M}}$) à considérer deux situations extrêmes correspondant respectivement à masse mini/inertie maxi et masse maxi/inertie mini.
- pour l'amortissement à considérer les valeurs minima telles qu'elles résultent des nombreuses mesures et essais effectués sur les ouvrages réels. C'est ainsi que pour les ouvrages métalliques, la valeur retenue est 0,5 %.

Il convient de noter cependant que la réponse dynamique de certains types d'ouvrages courants ou spéciaux est bien connue désormais du fait des constatations faites et des nombreuses mesures effectuées sur les lignes T.G.V. en exploitation. Leur caractéristique de raideur (tant en flexion qu'en torsion) et de masse font qu'ils se situent loin des zones de résonance ou que leur amplification dynamique sont très limitées. C'est le cas par exemple des caissons en béton précontraint qui ne font donc l'objet que de vérifications minimum.

Ce n'est pas le cas des ouvrages à ossature métallique qui font donc l'objet d'études dynamiques complexes en plus des études habituelles.

4.2 Etude du comportement dynamique de l'ouvrage

Ces études sont exécutées dès la phase projet à l'aide de programmes de calculs de structures (SAP, FASTRUDL, ANSYS...) qui permettent, sur la base de la méthode aux éléments finis, de résoudre l'équation du mouvement dynamique des poutres en flexion, en chaque point du système :

$$m(x)\frac{d^2y(t,x)}{dt^2} + c\frac{dy(t,x)}{dt} + \frac{d^2}{dx^2}\,EI(x)\,\frac{d^2y(t,x)}{dx^2} = p(t,x)$$

La méthode suivie consiste dans un premier temps à déterminer les modes successifs de vibration de la structure, puis à calculer la réponse de cette dernière par superposition modale en choisissant des vitesses de trains susceptibles de conduire à des situations de résonance (voir fig. 3 et 4).

Différentes modélisations ont été étudiées. Il en ressort qu'un modèle unifilaire avec caractéristiques torsionnelles est satisfaisant pour les ponts-mixte bi-poutres, mais qu'un modèle spatial est nécessaire dès que ce type d'ouvrage est biais. Dans le cas d'un modèle spatial, il s'agit de rendre compte avec précision de la flexion différentielle des poutres dans le comportement général en torsion de l'ouvrage.

Le chargement dynamique de la structure se fait noeud par noeud, à l'aide du logiciel spécifique prenant en compte le maillage du modèle. Il simule le passage à différentes vitesses d'une rame double du T.G.V. sur l'une ou l'autre des deux voies du tablier (voir fig. 5 et 6).

Statistiquement, le passage de deux circulations à très grande vitesse sur un tablier ,une par sens, ne génère pas d'amplification dynamique importante compte tenu des perturbations créées dans l'excitation. Elle est couverte, du point de vue résistance, par les calculs statiques majorés du coefficient dynamique réglementaire qui sont faits en chargeant les deux voies à l'aide du schéma de charges U.I.C. à 8 T/ml.

L'étude complète du comportement vibratoire d'un tablier comprend (voir fig. 7):
- une analyse modale de la structure ,
- la recherche de vitesses critiques pour lesquelles des phénomènes de résonance sont à craindre ,
- le calcul, à ces vitesses critiques ainsi qu'à des vitesses particulières de la ligne :

. de la flèche maximum au droit de chaque voie dans chaque travée ,
. des accélérations verticales dans l'axe de chaque voie ,
. des rotations des sections d'appui sur chaque culée ,
. du gauche de la voie.

4.3. <u>Etude du comportement dynamique du convoi</u>

Par ailleurs, la S.N.C.F. a développé un programme qui permet l'analyse dynamique d'un ou de plusieurs ensembles bogies/caisses au passage sur l'ouvrage en mouvement tel qu'il résulte des calculs de structures décrits ci-avant. Il est possible, en effet, à partir des résultats de l'analyse dynamique du pont, de calculer le déplacement $z_a(t)$ au droit d'un bogie donné et de calculer (voir fig. 8 et 9) le déplacement dans la caisse compte tenu des caractéristiques de sa suspension par intégration dans le temps de l'équation différentielle z(t) classique :

$$\frac{d^2z}{dt^2} + 2\xi\omega_n \left(\frac{dz}{dt} - \frac{dz_a}{dt}\right) + \omega_n^2(z(t) - z_a(t)) = 0$$

ω_n: pulsation de l'ensemble bogie/caisse.
ξ: ratio amortissement/amortissement critique

4.4. <u>Vérification à la fatigue</u>

L'historique des contraintes tel qu'il résulte de l'étude dynamique sous circulations réelles T.G.V. est utilisé pour calculer l'endommagement de la structure sous circulations réelles. Les règles utilisées sont celles de la C.E.C.M. (repris dans l'Eurocode 3). Ce calcul vient compléter celui, traditionnel et réglementaire, fait selon les règles de l'U.I.C.

Il faut toutefois signaler que cette vérification ne s'avère pas dimensionnante dans le cas général. En effet, compte tenu des règles de dimensionnement résultant des vérifications dynamiques, les ondulations de contrainte lors du passage d'une rame T.G.V. sont en général sous le seuil de troncature correspondant aux dispositions constructives retenues.

5. CONSEQUENCES DES ETUDES DYNAMIQUES SUR LA CONCEPTION DES OUVRAGES

Les conditions imposées en matière de déformation et d'accélération ($g<=0,35g$ pour le tablier et $g<=0,05g$ pour les caisses) imposent, pour les ouvrages mixtes bi-poutres auxquels se limite le présent exposé :

- une épaisseur de tablier voisine du 1/14è de la portée (à comparer au 1/18è pour les ponts sur lignes classiques) ;
- des épaisseurs de semelles surabondantes en travée vis-à-vis de la résistance, principalement sur les travées extrêmes des ouvrages continus qui doivent être aussi raides que les travées intermédiaires malgré, généralement, les différences de portée ;
- une épaisseur de dalle béton importante afin d'améliorer l'inertie de l'ouvrage et d'en augmenter la masse ;
- une rigidité torsionnelle importante obtenue par des pièces de pont de même hauteur que les poutres principales et espacées de 6 m (égal à l'écartement des poutres) et par un contreventement inférieur généralement en losange ;
- la limitation du biais des appuis.

6. CONCLUSION

Le passage à grande vitesse des groupes d'essieux nombreux et régulièrement espacés, tels qu'on les rencontre sur les rames T.G.V., induit, pour certains types d'ouvrages et dans certaines circonstances, des phénomènes de résonance qu'il convient de bien analyser si l'on veut bien maîtriser les problèmes de sécurité des circulations et de confort des voyageurs. Les nombreux essais effectués sur les lignes en cours d'exploitation à la S.N.C.F. jusqu'à des vitesses de 500 km/h, montrent que ces problèmes peuvent être résolus sans difficulté particulière. Les structures métalliques de type bi-poutre méritent cependant une attention spéciale en raison notamment de leur moindre masse, de leur faible amortissement et principalement de leur faible rigidité torsionnelle.

Figures diverses illustrant l'exposé sur le comportement vibratoire des ouvrages sous lignes à grande vitesse

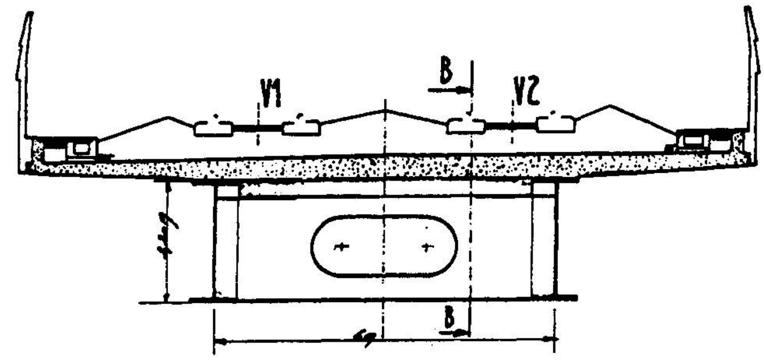

Fig. 1 : tablier bi-poutre acier béton

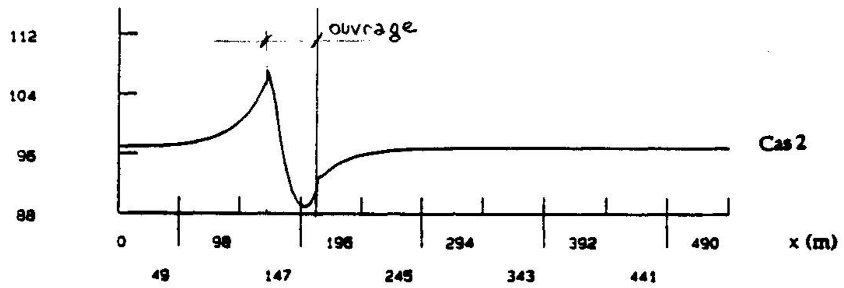

Fig. 2 : effort dans la voie sous effet thermique

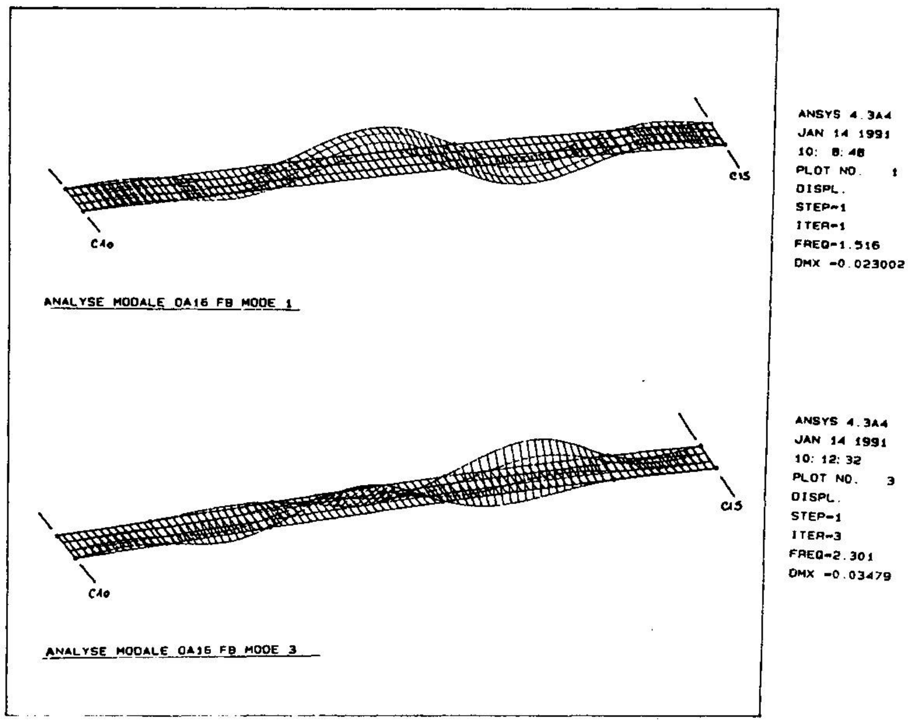

Fig. 3 : analyse modale tablier à 5 travées

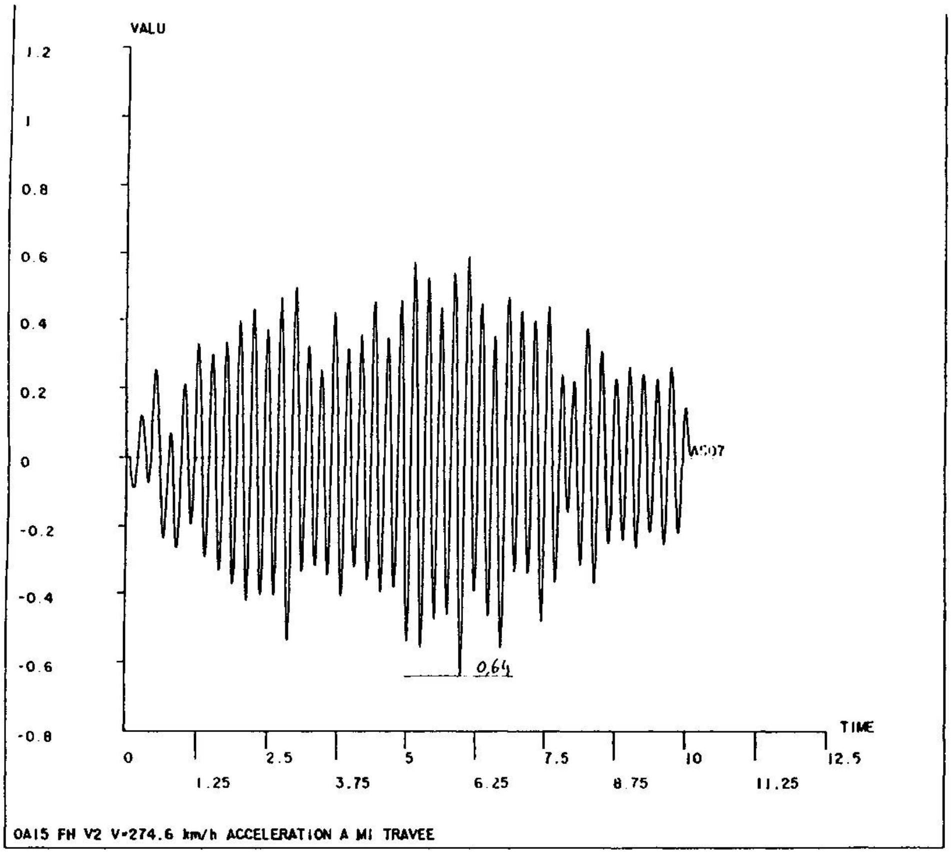

Fig. 4 : accélération en situation de résonance sur une travée intermédiaire

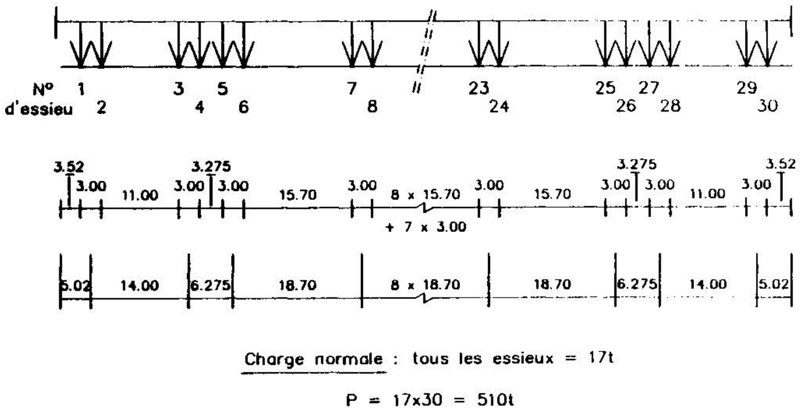

Fig. 5 : diagramme d'une rame double TGV

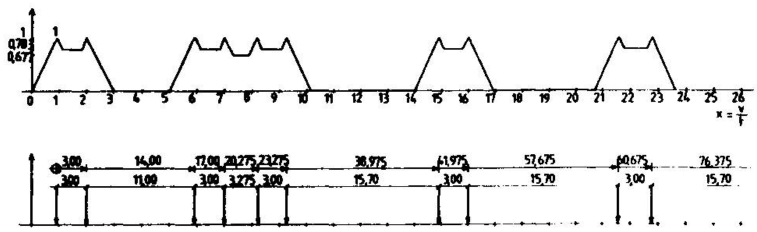

Fig. 6 : historique du chargement sur un noeud

VITESSE (km/h)	10	100	175	227.5	235.3	260	280	300.4	350	413.7	448.7
Accélération caisses (m/s2)	0	0.2	0.4	0.58	0.58	0.45	0.42	0.38	0.41	0.49	0.74
accélération ouvrage (m/s2)	0	0.5	1	1.85	2.21	1.4	1.14	1.14	1.36	2.83	2.52
flèches ouvrage (mm)	6.2	6.5	6.9	8.2	8.1	7.2	7.4	7.4	9.1	11.4	17.5
Rotation appuis (x10-3 rad)	0.29			0.53	0.58	0.4	0.36	0.32	0.34	0.41	0.56
Gauche rail (mm/m)	0.06			0.09	0.08	0.06	0.07	0.07	0.1	0.15	0.13

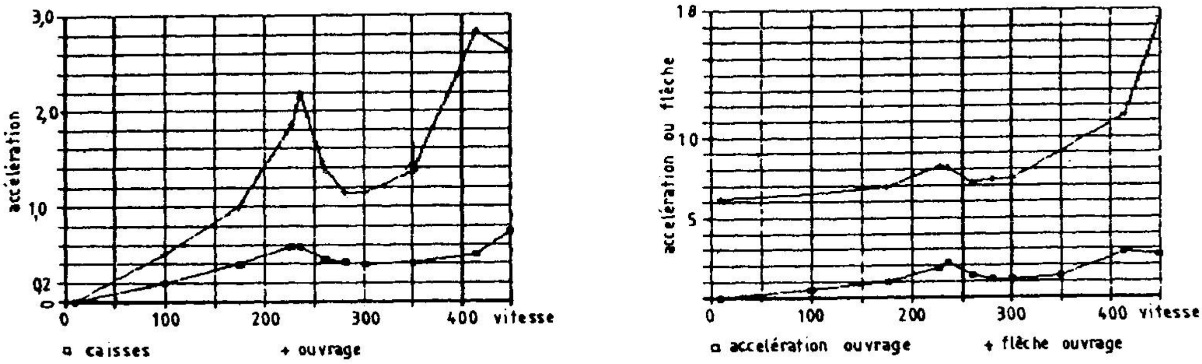

Fig. 7 : exemple d'analyse complète de comportement vibratoire

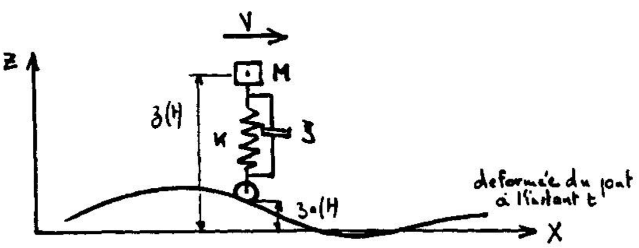

Fig. 8 : trajectoire des caisses - modélisation

N° DU BOGGIE	DEPL. VOIE (MM)	ACC. VOIE (M/S2)	DEPL. CAISSE (MM)	VIT. CAISSE (M/S)	ACC. CAISSE (M/S2)	IBLF
1	-5.065	.218	-5.261	-.025	.315	
2	-7.084	.374	-6.032	.028	-.297	
3	-6.851	.420	-5.207	.025	-.284	
4	-3.278	-.220	-1.839	-.009	-.115	
5	-2.342	.223	-2.587	-.012	.145	
6	-2.918	.260	-2.359	.011	.103	
7	-2.906	-.273	-2.625	.010	.145	
8	-3.104	-.190	-2.886	-.013	.159	
9	-3.040	-.208	-2.383	-.012	.136	
10	-3.462	.237	-2.401	.012	-.129	
11	-2.559	.213	-2.316	.011	.098	
12	-2.466	-.246	-2.129	-.010	-.134	
13	-4.309	.370	-4.207	.019	.235	
14	-4.990	.436	-4.613	.019	.246	
15	-5.643	.258	-4.679	.021	.224	
16	-5.289	-.297	-4.282	.018	.236	
17	-5.440	.331	-3.963	.018	-.229	
18	-5.038	.417	-3.786	-.018	.197	
19	-2.871	-.284	-2.517	.013	.129	
20	-2.101	.305	-1.721	-.007	.092	
21	-2.966	.351	-2.918	-.013	.152	
22	-3.184	.280	-2.220	-.012	.124	
23	-3.249	.460	-2.458	-.014	-.153	
24	-3.726	-.300	-3.533	.017	.169	
25	-2.851	-.280	-2.458	-.010	.132	
26	-2.459	-.325	-1.961	-.010	.113	
27	-2.725	.352	-2.716	-.016	-.209	
28	-6.965	.468	-5.944	-.026	-.327	
29	-7.333	.447	-5.990	.026	.298	
30	-5.770	.350	-4.301	.026	.310	

***** ANALYSE TERMINEE

Fig. 9 : exemple de comportement vibratoire des caisses à V 350 Km/h

Dynamic Problems on MAGLEV Guideway Structures

Problèmes dynamiques sur les structures de guidage à lévitation magnétique

Dynamische Probleme bei Magnetschwebebahn-Fahrwegen

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SUMMARY

This paper presents an introduction of the study for the design and technical development of a newly proposed guideway structure for the superconductive magnetically levitated railway system (MAGLEV). The design concept of the new guideway which combined twin precast prestressed concrete beams and infrastructures, and the dynamic design of structures for ultra-high-speed vehicles are described.

RESUME

Cette communication présente une introduction de l'étude sur la conception et la mise au point technique d'une structure de guidage récemment proposée pour le réseau ferroviaire à lévitation magnétique supraconductrice. Les auteurs exposent les caractéristiques essentielles du projet de ce système de guidage qui prévoit une double poutre préfabriquée en béton précontraint avec l'infrastructure correspondante. Ils fournissent une méthode de calcul dynamique des structures à prévoir pour les véhicules ultra-rapides.

ZUSAMMENFASSUNG

Der Aufsatz ist eine Einführung in die Entwurfstudien und die technische Entwicklung eines neuen Führungstragwerks für supraleitende Magnetschwebebahnen. Beschrieben werden das Konzept eines vorgespannten Zwillings-Fertigteilbalkens mit zugehöriger Ausrüstung und die Bemessung für dynamische Einwirkung aus dem Hochgeschwindigkeitsbetrieb.

1. INTRODUCTION

Extensive studies and developments are now being proceeded on the superconductive magnetically levitated railway system (MAGLEV) taking a new "side-wall levitation" as the fundamental system. All the ground coils including those for levitation, which was placed on the horizontal floor surface in the conventional system, are installed on the same vertical side-wall surface of the guideway. Although the section shape of the guideway remains the U-shape because the vehicles have to be supported by the floor slab at the time of wheel-traveling at low velocity, the new system has a lot of compositional advantages because the maintenance for the alignment of coils which require high accuracy can be conducted only by the side surface where the support, guidance and propulsion coils for the ultrahigh speed levitated-running are installed.
The authors proposed and have been developed a "Twin-Beam Guideway (TBG)" system in order to make the most of new system's advantage. The structural composition of the new guideway, and its dynamic behavior are reported in this paper.

2. STRUCTURE OF TWIN-BEAM GUIDEWAY SYSTEM

2.1 Composition of Twin-Beam Guideway

A schematic of TBG is shown in Fig.1. Its basic concept is as follows:
(1)The side surface finished to high accuracy is made of a precast concrete beam and set as a maintenance-free module with accurately aligned coils.
(2)The twin-beam (TB) can be placed on arbitrary infrastructures as both-ends supported beams. The structure of bearing is made finely adjustable in order to facilitate the correction of long-wavelength track irregularity.
(3)Separating the coil-installed module clearly from infrastructures, the coordination of inter-industrial works between structure, track and electricity is made easier and enabling a labor-saving, mechanized and rapid construction.

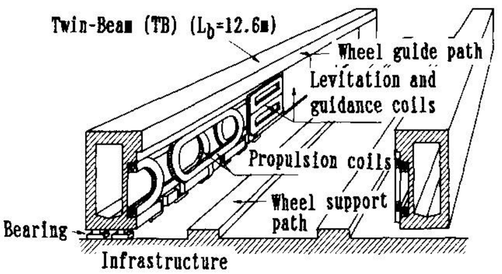

Fig.1 Twin-Beam Guideway

2.2 Structure of Twin-Beam

The sectional dimension of TB is shown in Fig.2. The determining factors for the dimension are as follows:
(1)Height is determined by the dimension of ground coils. Width is determined by the lateral rigidity and stability requirements for an independent beam. Length is set at 12.6 m to limit the deflection due to live load within 2 mm and considering the relation to the multiple of propulsion coil length (1.8 m).
(2)Box-beam is adopted except both-ends in order to obtain higher natural frequency.
(3)The pretensioning prestressed concrete structure is adopted to minimize the steel quantity in guideway and it is designed to keep full prestressing condition under service loads to sustain rigidity.

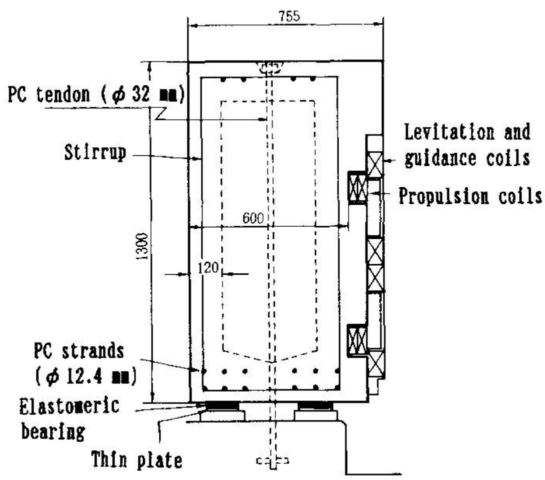

Fig.2 Cross section

2.3 Structure of bearing

The structure of bearing is shown in Fig.3 and the technical key points are as follows:
(1)Elastomeric bearings with lead cylinder at center are adopted for economic efficiency and requirements to provide enough horizontal spring constant. They allow free expansion and contraction of beam under the slow-strain loading due to temperature change.
(2)PC tendons are placed in order not to produce negative reaction at supports due to the eccentric loading of vehicle and large horizontal loading at emergency.

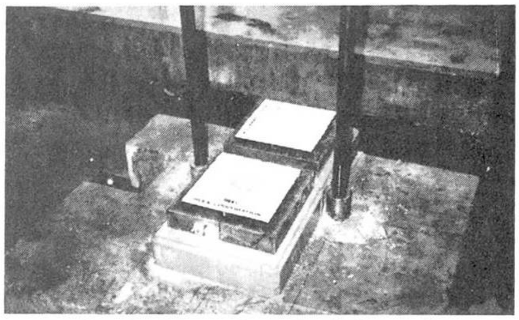

Fig.3 Structure of bearing

3. DYNAMIC RESPONSE OF GUIDEWAY FOR ULTRA-HIGH VELOCITY

The study of dynamic response of guideway is very important from the point of view not only of the structural design but also of the riding comfort because the dynamic load due to vehicle may determine the design condition and its deflection may work as a track irregularity.
As JR's MAGLEV is adopting a system of concentratedly-arranged superconductive magnets to minimize the magnetic field in the cabin, the train load becomes a series of distributed loads with a constant interval as shown in Fig.4 so that it resonates the guideway at a certain velocity. An analytical assessment of dynamic response of the TB and bridges which support them is made in this chapter.

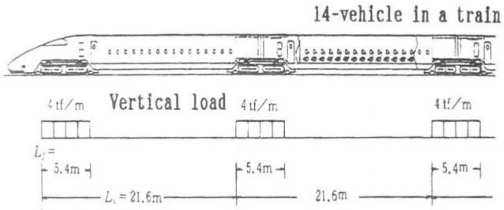

Fig.4 MAGLEV train load

3.1 Theory and Analysis of Dynamic Response

3.1.1 Equation of motion.
The equation of motion of a simply supported beam may be resolved into independent one degree of freedom systems with respect to the modes of vibration in the fundamental equation of motion of Bernoulli-Euler for the forced vibration of viscous-damped system approximating the deflection to the summation of principal mode functions as shown in Eq.(1).
Focusing on the first mode of vibration which has a dominant influence on the dynamic response of beam, the non-dimensional equation of motion as shown in Eq.(2) may be derived for a series of unit forces acting with a constant velocity and at a regular interval.[1,2]
The dynamic response of beam is governed by two dominant factors α and L_b/L_v, but the response value is obtained taking into account the additional factors ζ and N.

$$y = \sum_{i=1}^{n_b} \phi_i \cdot \sin(i\pi x/L_b) \quad (i=1,2,3,\ldots,n_b) \quad \ldots(1)$$

where, y:the deflection of beam, x:the distance from an end of beam, L_b:the span length, n_b:the mode number of beam.

$$\ddot{\phi} + \frac{2\zeta}{\alpha}\dot{\phi} + \frac{1}{\alpha^2}\phi = \frac{1}{\alpha^2}\sum_{j=1}^{N}\varepsilon_j \cdot \sin(\tau-\tau_j) \quad \ldots(2)$$

Where, $\tau_j=(j-1)\cdot\pi\cdot(L_b/L_v)^{-1}$

$$\varepsilon_j=\begin{cases}1 : 0 \leq \tau-\tau_j \leq \pi \\ 0 : \tau-\tau_j < 0 \text{ and } \tau-\tau_j > \pi\end{cases}$$

where, τ :the non-dimensional time (the duration which a load passes through the beam is defined as π), α :speed parameter,

$\alpha = v / (2 f_1 \cdot L_b)$ ... (3)

where, v:the velocity of vehicle (m/s), f_1:the fundamental natural frequency of beam (Hz).

L_b/L_v:the non-dimensional span length of beam (L_v:interval of loads),

N:the number of loads, ζ :the damping ratio of beam.

3.1.2 Dynamic response of simply supported beam

Substituting the MAGLEV train load (see Fig.4) into the term of excitation in right hand side of Eq.(2), the dynamic load factors, λ, (DLF=the ratio of the maximum dynamic deflection to the maximum static deflection at the center of span by the application of loads) with respect to non-dimensional span length of the beam are obtained. The influence of distributed load is taken into account in the computation. When the load excites the beam with constant period, the beam is resonated at a certain velocity and generates peaks of DLF. The relations between λ and L_b/L_v are shown in Figs.5 and 6.

3.1.3 General law of resonance

The following laws may be derived on the resonance.[3]

(1)The velocity at the primary resonance where the DLF becomes the maximum is given by Eq.(4) using the speed parameter. Peaks of DLF also appear at one-half, one-third and so on of the primary resonant velocity.

$\alpha = L_v \sqrt{1-\zeta^2} / (2L_b)$ ... (4)

(2)The value of DLF as shown in Figs.5 and 6 is governed by Lb/Lv, and influenced by ζ, L_f/L_v and N. (L_f:length of distributed load)

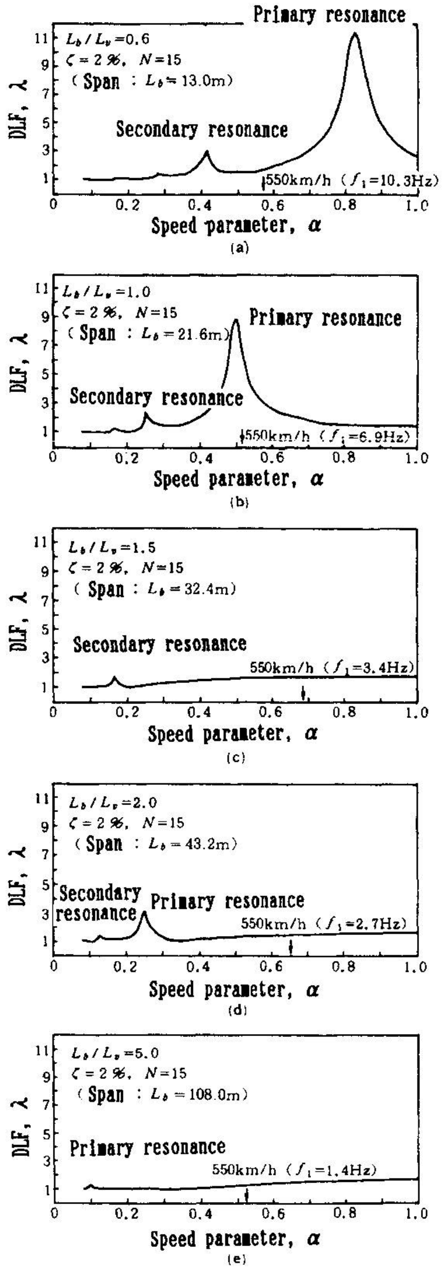

Fig.5 Dynamic response of simply supported beam

(3)When the speed parameter which is determined by Eq.(4) coincides with that of Eq.(5), the resonance does not occur. This is a singularity where no residual vibration is produced in undamped vibration system. The effect of repetition due to sequential loading disappears.

$\alpha = 1 / (2k+1)$ (k=1,2,3,...) ...(5) $L_b/L_v = k+0.5$ (k=1,2,3,...) ...(6)

When the damping ratio is sufficiently small ($\zeta \fallingdotseq 0$), the singularity of disappearing resonance is expressed by the relation between the span and vehicle lengths as Eq.(6). As mentioned above, the dynamic response and resonance of simply supported beam by the sequential distributed load with intervals are clarified. However, the damping ratio has to be studied further to compute the appropriate DLF for the design of structure.

3.2 Dynamic Response of Bridges and Determining Factor in Design

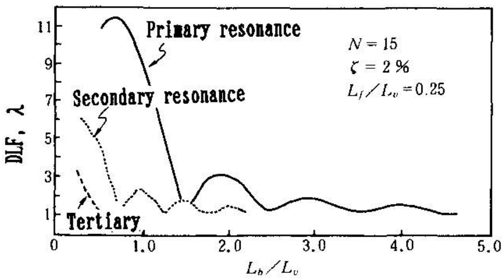

Fig.6 DLF at resonance

Although the span lengths of bridges which support the TB widely range from approximately 10 to more than 100 meters, the dynamic response and the determining factor for structural design[4] may be classified by the dominant parameter L_b/L_v based on the analysis and the law of resonance as follows. (As for the values of f_1, which are necessary to convert α into the practical velocity, the standard values for bridges on the Shinkansen-line are used as appropriate ones.)

(1)When L_b/L_v is 0.6 (see Fig.5), the design impact factor for the assessment of ultimate limit state may be determined by the DLF of secondary resonance which appears at approximately 400 km/h.
(2)When L_b/L_v is 1.0, the primary resonance appears just around 550 km/h. If the fundamental natural frequency of bridge can be shifted higher, the maximum DLF may be avoidable. On the other hand, it can be designed to cover the primary resonance, but, careful assessments of ultimate, serviceability and fatigue limit states are essential in this case.
(3)When L_b/L_v is 1.5, α of the primary resonance coincides with one-third. It is the singularity of disappearing resonance. Only a small peak of secondary resonance is observed. Bridges can be designed by the smallest live load.
(4)When L_b/L_v is 2.0, the velocity at the primary resonance appears much lower than 550 km/h. The design impact factor for the assessment of ultimate limit state has to be determined by the DLF of the primary resonance, but those for the serviceability and fatigue limit states may be determined by the DLF at the service velocity of the train.
(5)When L_b/L_v is 5.0, the influence of the resonance is very small. The DLF becomes almost equal to that of the speed-effect of a single load. The allowable limit of deflection for the train load may become the determining factor in the design.

3.3 Dynamic Response of Twin-Beam

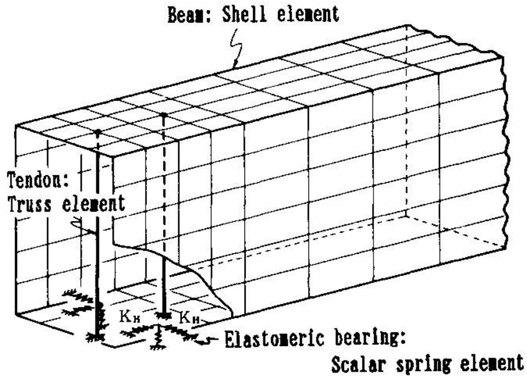

Fig.7 Analytical model of TB

More detailed study on the dynamic response of TB is necessary for the following reasons:
(1)Torsional moment with vertical load acts due to the eccentric loading of vehicle.
(2)As the TB is supported by elastomeric bearings, the dynamic response may change depending on the elasticity of the bearings.

The dynamic response of the beam to the passage of MAGLEV train load (5 vehicles) is computed using a simulation program which can model the structure by 3D-FEM. Only vertical loads are used as the train load in this case.
A schematic of analytical model of the TB is shown in Fig.7. The beam is modeled by approximately 600 shell elements. Two cases are treated in the analysis using different spring constants for the elastomeric bearing with and without lead cylinder (K_H=9500tf/m, 2000tf/m, respectively). The response waves of vertical and horizontal deflection of the TB at 550 km/h at the center of span, and the relation between velocity and maximum response values are shown in Figs.8 and 9, respectively.
In the vertical direction, the fundamental natural frequency is approximately 17 Hz. The TB can be assumed to behave as a simply supported beam on rigid foundation judging from the magnitude of deflection. As α at 550 km/h is as small as 0.37, little incremental tendency in DLF is observed.

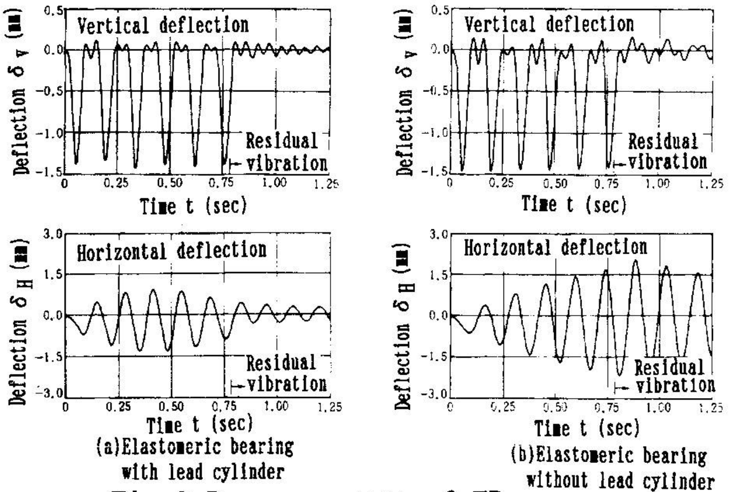

Fig.8 Response wave of TB (at center of span: 550 km/h)

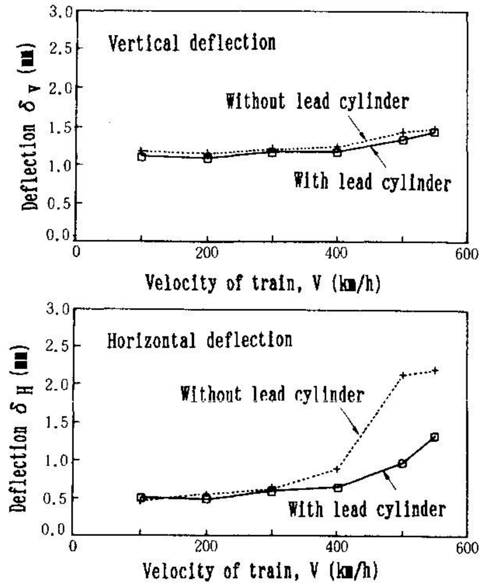

Fig.9 Relation between velocity of vehicle and maximum response

In the horizontal direction, quite different responses are observed for the cases with and without the lead cylinder in elastomeric bearing. In the case with lead cylinder, the maximum response at 550 km/h stays approximately at 1.4 mm and convergency of response with respect to the number of vehicles seems stable. The DLF, however, has a slight increasing tendency. In the case without lead cylinder, the response seems to diverge at 550 km/h.

The relation between fundamental natural frequency and spring constant of horizontal direction is shown in Fig.10. K_H needs to be more than 7000 tf/m. When K_H is small and f_1 is less than 8 Hz, a divergency seems to appear.

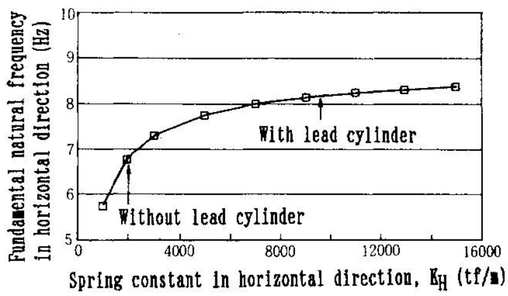

Fig.10 Fundamental natural freq. of TB in the horizontal direction

4. CONCLUSIONS

The knowledge obtained through this study is summarized below.

(1)The Twin-Beam can be manufactured very accurately and utilized as a maintenance-free module-structure with ground coils.

(2)The dynamic response with respect to the velocity of train load and the determining factors for the structural design are clarified for the concrete bridges which support the Twin-Beam.

(3)The analytical study using 3D-FEM reveals that the Twin-Beam has sufficient rigidity in the vertical and horizontal directions and shows stable dynamic response with respect to velocity of train. The premise, however, is that the horizontal spring constant of bearing be made large enough by using the elastomeric bearing with lead cylinder.

5. REFERENCES

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Fatigue and Life Prediction for High-Speed Railway Bridges

Fatigue et durée de vie restante pour les ponts ferroviaires à grande vitesse

Ermüdungs- und Restlebensdauer-Prognose für Hochgeschwindigkeits-Eisenbahnbrücken

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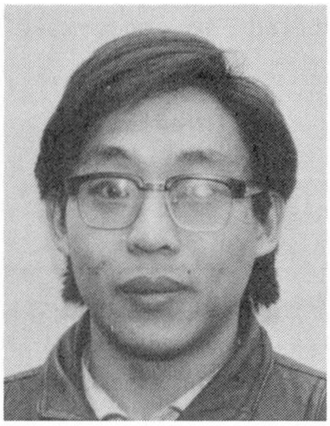

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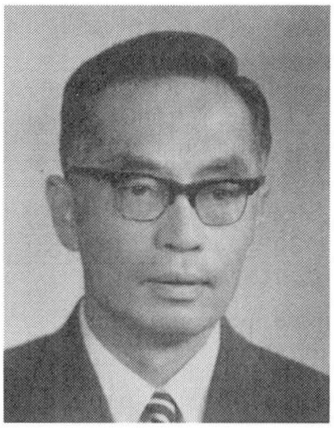

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SUMMARY

The authors discuss several problems associated with fatigue damage assessment and remaining life prediction.

RESUME

Les auteurs examinent quelques uns des problèmes relatifs à l'évaluation des dommages dus à la fatigue et à l'estimation de la vie restante.

ZUSAMMENFASSUNG

Einige spezifische Probleme, welche mit der Auswertung der Ermüdungsschädigung und der Prognose der Restlebensdauer verbunden sind, werden diskutiert.

1. INTRODUCTION

High-speed railway is the main tendancy of modern railway transportation. In France and Japan, vehicle speed of high-speed railway has reached about 300 to 400 Km/h. In China, decision has also been made to develop quasi-high-speed railway (about 160 to 200 Km/h) in recent years on some busy railway lines such as the Guangzhou-Shenzhen line. CARS and other railway institutions are doing site investigation of the existing tracks and bridges to assess their dynamic responses and remaining strength capacities at the speed of 160 Km/h.

The explicit effects of high-speed train on bridge structures are the larger impact and the higher fatigue frequency. Hence, high-speed railway bridge undertakes more serious fatigue effects than that on common lines. So we must pay more attention to fatigue problems of structures and do research on the fatigue behavior of high-speed railway bridges. In this paper, the authors will discuss several important problems associated with fatigue damage assessment and remaining life prediction of existing bridges which will be modified to undertake high-speed transportation.

2. FATIGUE DAMAGE AND S-N CURVE

According to Damage Theory by Lemaitre, we have

$$\underset{\sim}{\sigma} = (1-D)\ \underset{\sim}{E} : \underset{\sim}{\varepsilon}^{e}$$

$$\dot{D} = A\ y^{s} \dot{p}$$

$$y = \frac{\sigma_{eq}^{2}}{2(1-D)}\left[\ 2/3(1+\mu) + 3(1-2\mu)\left(\frac{\sigma_m}{\sigma_{eq}}\right)^2\right]$$

where $\underset{\sim}{E}$ is elastic tensor of undamaged material, y is called strain release rate of damage, σ_m is average stress, and σ_{eq} is Von-Mises equivalent stress.

Using Ramberg-Osgood hardening rule in case of multiaxial stress state, it can be obtained [3]

$$\dot{D} = B\, \tilde{\sigma}_{eq}^{\beta}\, \dot{\tilde{\sigma}}_{eq}$$

where $\tilde{\sigma}_{eq} = \sigma_{eq}/(1-D)$

$$B = A/2^{S}\left[2/3(1+\mu) + 3(1-2\mu)\left(\frac{\sigma_m}{\sigma_{eq}}\right)^2\right]^{S} M/K^{M}$$

$$\beta = 2s + M - 1$$

In case of proportional loading, B and β are constant, and the following equations can be obtained,

for symmetrical stress fatigue (ρ =-1)

$$\frac{\delta D}{\delta n} = 2B/(1+\beta)/(1-D)^{1+\beta}\ [\Delta\sigma_{eq}/2]^{1+\beta} \qquad (1)$$

for pulse stress fatigue (ρ =0)

$$\frac{\delta D}{\delta n} = B/(1+\beta)/(1-D)^{1+\beta}\ 2^{1+\beta}\,[\Delta\sigma_{eq}/2]^{1+\beta} \qquad (2)$$

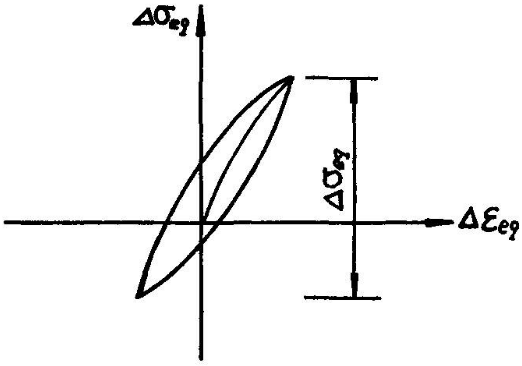

Fig.1 Stress-Strain Cycle

In case of constant range fatigue, we obtain

$$D = 1-(1-n/N)^{\frac{1}{2+\beta}} \qquad (3)$$

$$N(\Delta\sigma_{eq}/2)^{1+\beta} = \begin{cases} (1+\beta)/2/B/(2+\beta) & \text{for } \rho=-1 \\ (1+\beta)/2^{1+\beta}/B/(2+\beta) & \text{for } \rho=0 \end{cases}$$

where n represents number of fatigue cycle, and N is the number of cycles at fatigue failure of material.

These equations demonstrate fatigue damage and fatigue life behaviors of materials respectively

Therefore, S-N relation for a real component of bridge can be expressed experimentally as

$$N(\Delta\sigma_{eq}/2)^{\alpha} = C \qquad (4)$$

and the damage accumulating rule for such a component can be described with equation (3), in which D behaves nonlinearly with (n/N)

3. NONLINEAR DAMAGE ACCUMULATING RULE AND REMAINING FATIGUE LIFE

3.1 Nonlinear Damage Accumulating Rule

In case of variable range fatigue, from equation (3) we get

$$\delta D = 1/(2+\beta)/(1-D)^{1+\beta}\delta(n/N) \qquad (5)$$

For a definite loading history (n_i, N_i), equation (5) can be integrated into

$$D = 1-[1-\sum_i(n_i/N_i)]^{\frac{1}{2+\beta}} \qquad (6)$$

According to Miner's rule

$$D_M = \sum_i (n_i/N_i)$$

So D and D_M have following relation (see Fig.2),

$$D = 1-(1-D_M)^{\frac{1}{2+\beta}}$$

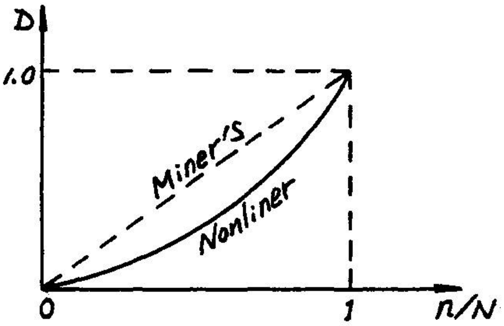

Fig. 2 Fatigue Damage Accumulating

3.2 Remaining Fatigue Life

In case of constant range fatigue, assuming D_0 represents fatigue damage at the moment from which the remainig life is considered, we can obtain the remaining fatigue life as

$$n' = (1-D_0)^{2+\beta} N$$

In case of variable range fatigue, we assume that loading spectrum per year is unchanged, hence

$$D_0 = 1-[1-N_p\sum_i (n_i/N_i)]^{\frac{1}{2+\beta}}$$

where N_p represents fatigue history passed (in year). Therefore the remaining fatigue life can be obtained as

$$n' = (1-D_0)^{2+\beta}/\sum_i(n_i/N_i)$$

4. FATIGUE DAMAGE ASSESSMENT OF EXISTING BRIDGES

Assessment approach of fatigue damage for existing structures has been discussed in many papers [1,2,5]. In this paper, the authors will suggest a method based on the quasi-static concept.

From continuum damage mechanics,

$$D = 1-\sigma/\tilde{\sigma} \doteq 1-(\Delta\varepsilon_0/\Delta\varepsilon)$$

If the initial strain range ,$\Delta\varepsilon_0$, and the present strain range,$\Delta\varepsilon$, are known, D can be obtained. Generally, it is difficult to get $\Delta\varepsilon_0$. However, we can calculate D by using the results $\Delta\varepsilon_1$ and $\Delta\varepsilon_2$ from two times of experiments.

$$(1-D_1)/(1-D_2) = (\Delta\varepsilon_2/\Delta\varepsilon_1)$$

From equation (6)

$$D_1 = 1-[1-N_p\sum_i(n_i/N_i)]^{\frac{1}{2+\beta}}$$

$$D_2 = 1-[1-(N_p+N)\sum_i(n_i/N_i)]^{\frac{1}{2+\beta}}$$

where N is the time interval (in year) between two times of investigation.So N_p, D_1 and D_2 can be assessed and calculated.

If load spectrum in remaining time is ($\tilde{n}_i$, $\tilde{N}_i$), then

$$D_1 = 1-[1-N_p'\sum_i(\tilde{n}_i/\tilde{N}_i)]^{\frac{1}{2+\beta}}$$

Therefore the remaining life is

$$n' = (1-D_1)^{2+\beta}/\sum_i(\tilde{n}_i/\tilde{N}_i)$$

Analytical flowchart for damage assessment and remaining life prediction for existing bridges is shown in Fig. 3.

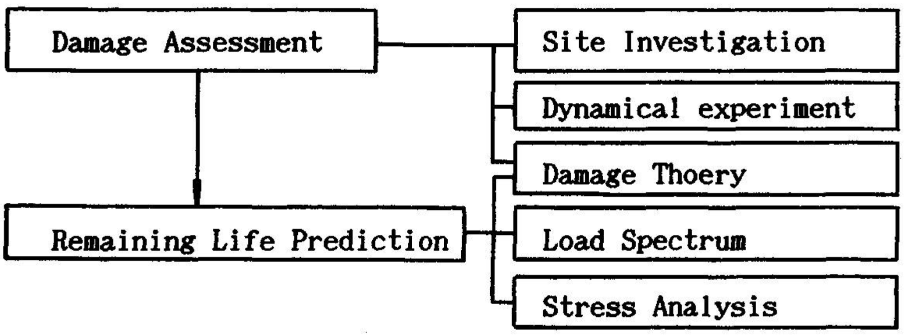

Fig.3 Analytical Flowchart

For high-speed railway bridges, more attention should be paid to load spectrum.

5. AN EXAMPLE

Sketch of Puyangjiang River Railway Bridge, situated on Zhejiang-Jiangxi Line, is shown in Fig.4. The 32m plate girder was fabricated in 1972. The stress spectra of the longitudinal girder and the cross beam were obtained by CARS in 1983.

The S-N curve for welded component proposed by AREA was chosen to predict fatigue damage and remaining fatigue life of the plate girder. The analytical results are listed in Tab.1 and Tab.2.

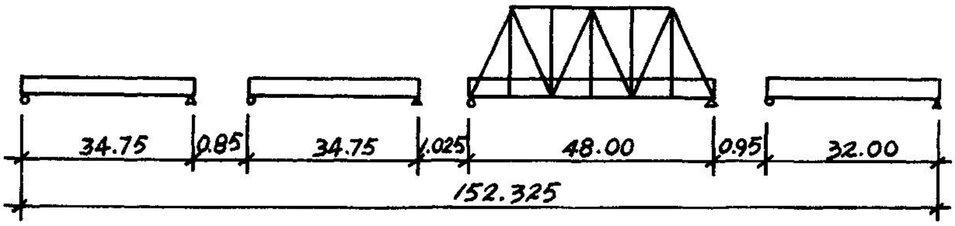

Fig.4 Sketch of Puyangjiang River Bridge

Table 1. Fatigue damage at the lifetime of 100 years

	Main girder	Longitudinal girder	Cross beam
D	1.89	0.623	0.235
D		0.216	0.065

Table 2. Total fatigue life and remaining life (in year)

	Main girder	Longitudinal girder	Cross beam
Total	52.9	160.5	425.5
Remaining	34.9	142.5	407.5

6. CONCLUSIONS

Fatigue damage assessment and remaining fatigue life prediction are very important subjects for existing structures,especially for high-speed railway bridges. Based on continuum damage mechanics, a nonlinear damage accumulating rule, damage assessment approach and remaining life prediction method have been proposed and proved by field tests on Puyangjiang River Bridge in this paper.

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Long Span Bridges of the New Railroad Lines in Germany

Ponts à grande portée des récentes voies ferroviaires en Allemagne

Weitgespannte Brücken der Neubaustrecken der Deutschen Bundesbahn

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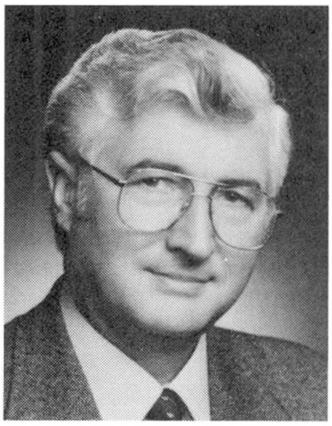

Wilhelm Zellner, born 1932, received his civil engineering degree at the University of Vienna, Austria. For two years he was in charge of the supervision of a big prestressed concrete viaduct in Vienna. In 1962 he moved to Leonhardt & Andrä to Stuttgart and in 1970 became a partner in this firm.

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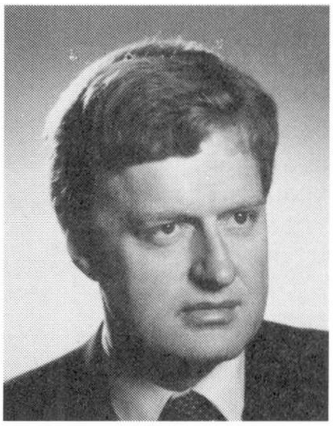


Reiner Saul, born in 1938, received his civil engineering degree at the University of Hannover, Germany. Four years with a steel contractor, since 1971 senior supervising engineer with Leonhardt, Andrä & Partner. He was responsible for the design, technical direction and checking of numerous long span bridges, including also major rehabilitation works.

SUMMARY

The German Federal Railway Authority built two new railroad lines with a total length of 427 km in the years 1980 to 1991. These lines are designed for speeds of 250 km/h, which is the envisaged speed for the future, also on other lines. The topography is hilly and the new alignment is rather straight. Therefore about 10% of the new lines are on bridges and 30% in tunnels. Six special bridges with spans longer than 75 m had to be designed and built, which are briefly described.

RESUME

Les Chemins de Fer Allemands ont construit au cours des années 1980 à 1991 deux nouvelles voies de 427 km de longueur, sur lesquelles les futures générations de trains rapides doivent pouvoir circuler à une vitesse de 250 km/h. La topographie du site est mouvementée. Le profil en long est plutôt rectiligne, donnant lieu à un pourcentage en ponts et tunnels par rapport à la longueur totale du tracé de 10 et respectivement 30%. Six ponts spéciaux d'une portée de plus de 75 m ont été étudiés et construits. Ces ponts font l'objet de la présente communication.

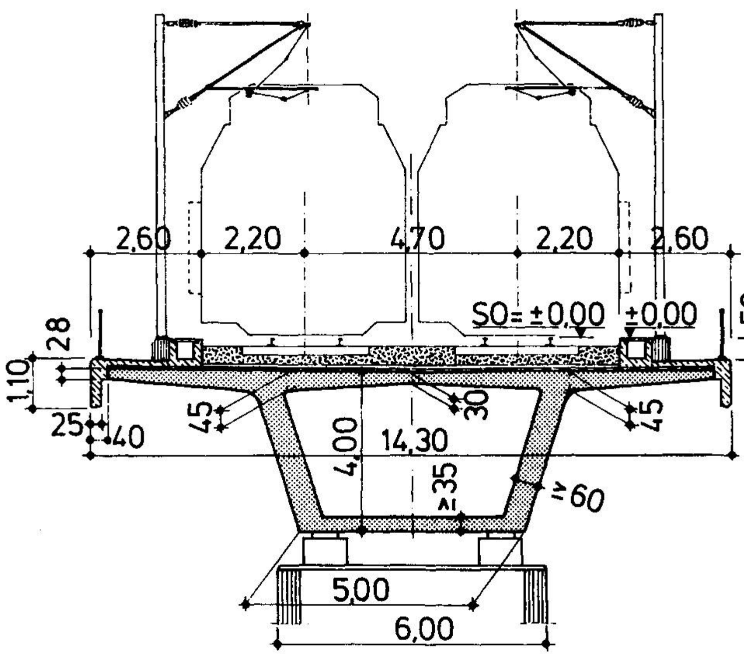
ZUSAMMENFASSUNG

In den Jahren 1980 bis 1991 baute die Deutsche Bundesbahn zwei neue, zusammen 427 km lange Eisenbahnlinien für den 250 km/h schnellen Verkehr der Zukunft. Wegen der topografisch schwierigen Mittelgebirgslandschaft laufen die grosszügig trassierten Strecken zu etwa 10% ihrer Gesamtlänge über Talbrücken und zu 30% in Tunnel. Sechs Sonderbrücken mit Spannweiten von über 75 waren zu planen und zu bauen, worüber hier berichtet wird.

1. INTRODUCTION

The New Railroad Lines for the fast trains require many bridges and viaducts. Due to the hilly topography, about 10 % of the total length of the lines are bridges in Germany. Most of the viaducts have been built according to the so-called "General Design" (Rahmenplanung) by simple supported concrete box girders with a distance of the piers of 44m or 58m and a depth of the girders of 4.oo m or 5.3o m resp. (Fig.1).

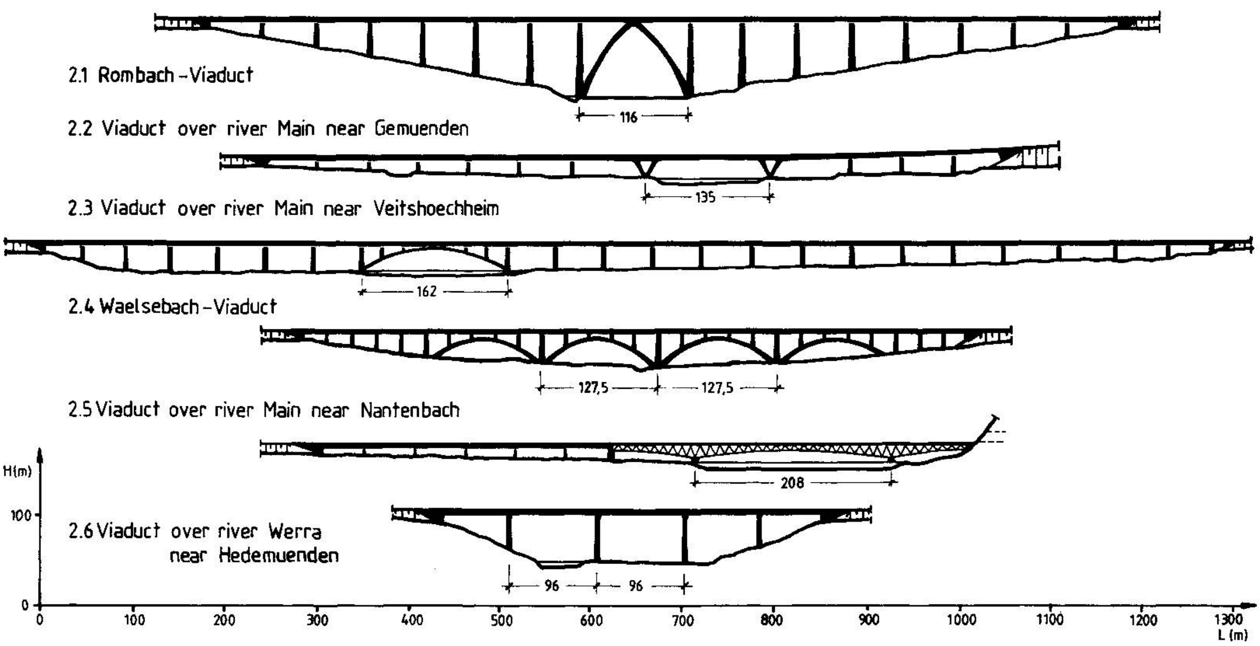
Fig.1 Standard cross-section of simple supported girders ℓ = 44m



Such bridges can be built with joints between the girders but without joints in the rails and in ballast, if the piers are not higher than 20 to 25m. In this case the big horizontal braking forces can be transmitted to the ground by the piers. If the piers are higher than 25m, then the piers become too flexible and the braking forces would be transmitted via the rails to a stiff pier or a stiff abutment. In this case the stress in the rails from the big longitudinal force becomes too big and one has to find other solutions for the transmission of the braking force.

High viaducts or long spans over rivers can not be built with jointless rails. Rail expansion joints are developed and in use in Germany for expansions up to 800 mm, that means the fixed point of the bridge may be in a distance up to

Fig. 2: Long Span Bridges of the New Railroad Lines in Germany (Spans >75m)



650 m from this expansion joint. In consequence of this basic considerations, several bridges had to be designed which are beyond the "General Design", because of its span (>58m) or its height above ground (>25m).
Six bridges have spans of more than 75m. These bridges are called the long span bridges and this article will deal with them. Three of them have been designed by Leonhardt, Andrä & Partner (Gemünden, Veitshöchheim, Nantenbach). One bridge has been preliminary designed by this consulting firm (Hedemünden). Two of them have been designed by others (Rombach and Waelsebach Viaduct, Fig.2).

2. ROMBACH VIADUCT (Fig.2.1 and Fig.3)

The total length of this bridge is 17 x 58 = 986 m. The maximum height of the rails above the valley is 94m. It was not possible to transmit the braking forces via vertical piers to the ground. If the bridge would have been fixed to one abutment, the distance to the rail expansion joint would have been more than 650 m. Therefore, the superstructure has been fixed at the center of the bridge above a huge concrete A-shaped trestle. Two rail expansion joints have been provided, one each above the abutments. The legs of the trestle are curved in order to counteract the dead weight of the leg.

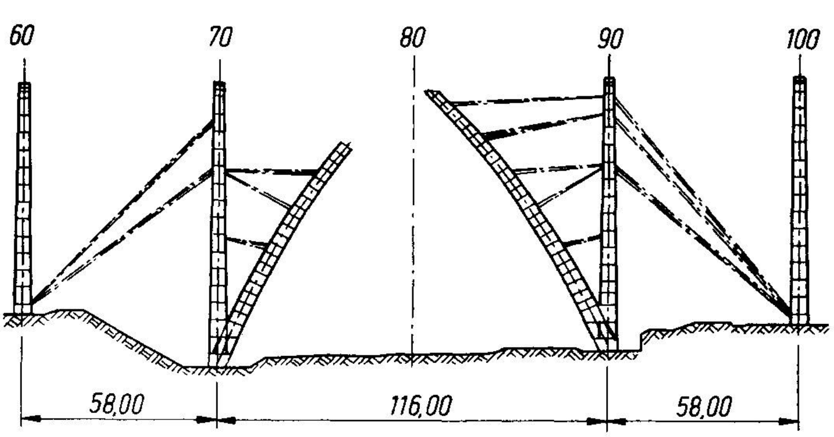


Fig. 3 Construction of concrete A-trestle

The superstructure consists of 17 simple supported girders with spans of 55.75 m and a depth of the girders of 5.3o m. The girders are longitudinally coupled in the center of gravity of the cross-section. The couplers consist of 8.2o m long tendons for the transmission of longitudinal tension forces and neoprene bearing pads for the transmission of compression forces.

Should it ever be necessary to replace the superstructure by a new one, the work can be done in 58m long parts. It is assumed that a new simple supported girder will be built aside of the old one on auxiliary piers and the replacement can be done within two days by simultaneous lateral shifting. The same system with an A-shaped trestle in the middle of the total bridge length has been used for two more bridges: the Mülmisch and the Pfieffe Viaduct with 15 x 58 = 870 m and 14 x 58 = 812 m length, resp. The A-shaped trestles for those bridges have been built by the use of a total formwork, resting on a huge wooden scaffolding. The superstructure of these 3 bridges have been built by a special formwork, bridging the full length of a span of 58m and launched from span to span [1].

3. VIADUCT OVER THE RIVER MAIN NEAR GEMÜNDEN (Fig.2.2 and Fig.4)

This viaduct consists of a 299 m long main bridge over the river Main, a 330.5 m long northern approach bridge and a 164 m long southern approach bridge. The approach bridges have been built by the incremental launching method. The main bridge was built by the free cantilevering construction method. The longitudinal horizontal forces (from braking and from the friction at bearings) are transmitted to the abutments. The braking forces, acting onto the main bridge are transmitted to the ground by frame action of the double V-shaped struts of the frame. Under the V-shaped struts are concrete hinges for working forces up to 123 MN.

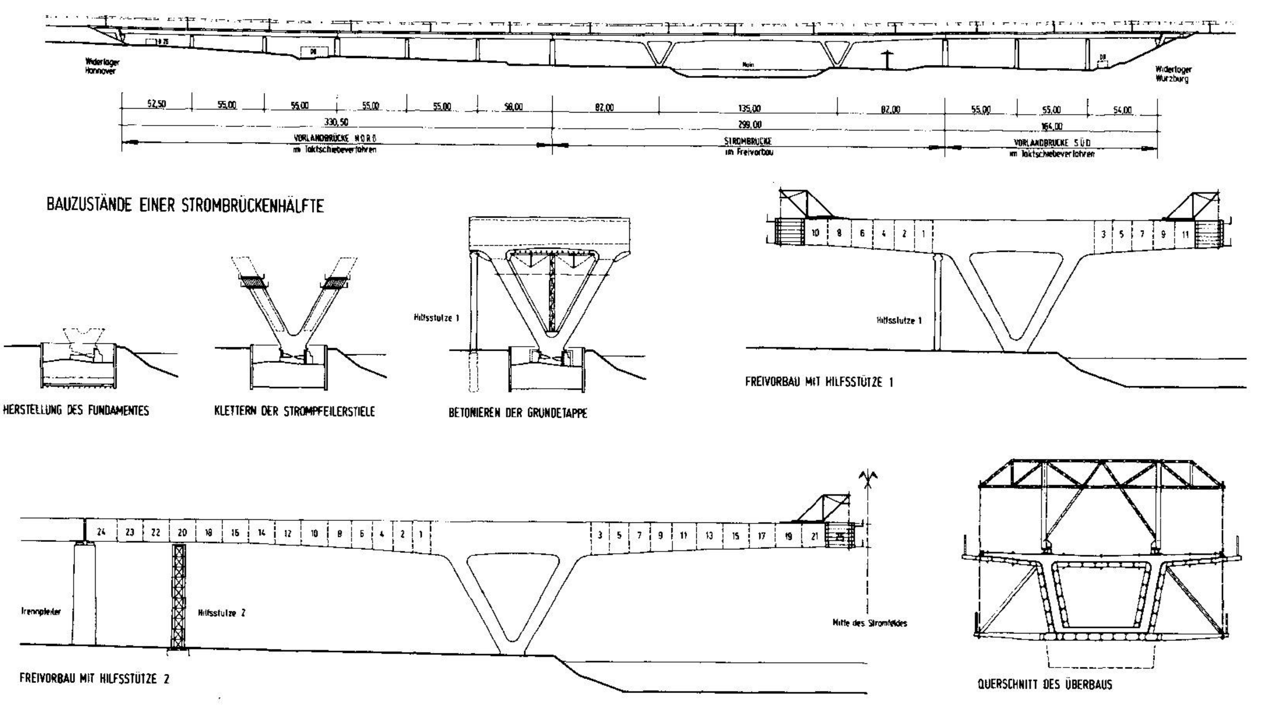


Fig. 4 Viaduct near Gemünden: Overall view and construction

Should it ever be necessary to replace the 299 m long main bridge, a new bridge will be built aside of the old one. The concrete hinge will be cut, one inch deep, then broken by lifting the bridge with jacks about 15 mm. The replacement will be done by simultaneous lateral shifting. The hinge will be repaired by epoxy mortar. The rail traffic will be interrupted by about three days.

The two rail expansion joints are at the ends of the main bridge. The depth of the girder is 4.5o m in general, but 6.5o m above the V-shaped struts. Without the frame action the beam over a span of 135 m would have been min. 9.5o m deep for the heavy live loads from the trains (80 kN per lin.m per track) and the dead load from the ballast and the concrete structure itself.

There has been too much opposition from the public against the 9.5o m deep beam close over the bottom of the valley. The frame solution, therefore, is considered to be an essential improvement with regard to the aesthetics [2], [3].

4. VIADUCT OVER THE RIVER MAIN NEAR VEITSHÖCHHEIM (Fig.2.3 and Fig.5)

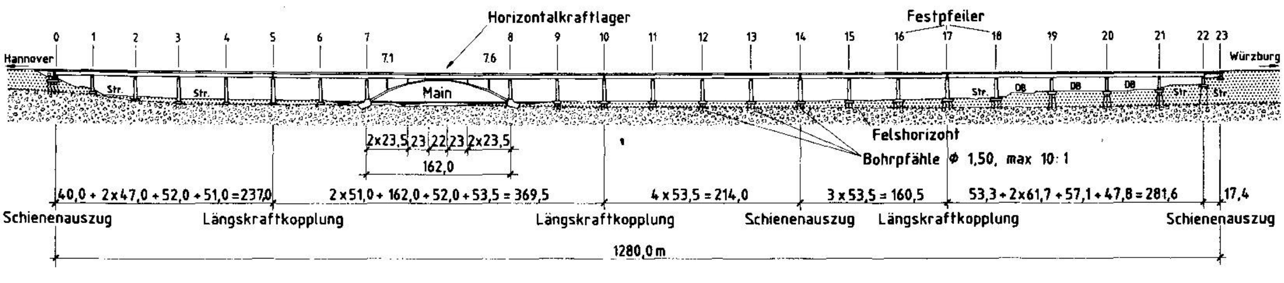


Fig. 5 Viaduct near Veitshöchheim: Elevation view

This bridge is explained in detail in the report on the Poster Sessions W1.

5. WAELSEBACH VIADUCT (Fig.2.4 and Fig.6)

The New Railroad Line from Hannover to Würzburg crosses the Waelsebach valley on a 721.2 m long viaduct in a maximum height of 40m above the ground of the valley. The alignment is curved in plan with a radius of 5100 m and the gradient in elevation is 1.2458 ‰. The soil under the bridge generally consists of solid sandstone and limestone. At four locations this rock formation has fallen into deep caves and the gaps in the rock, called chimmneys, have been filled with soft clay. The gaps are up to 60m wide and could not been used to carry foundations. Two gaps near the abutments have been bridged by earth-covered concrete box girders, so-called chimney-bridges. Two other gaps have been bridged by visible concrete arches. Thus the bridge appears with 4 main arches at spans of 127.5 m each. Piers have been built at a distance of 25.5 m for the approach bridges and on the arches. The piers and the two abutments support 28 simple supported girders, thus the total length of the bridge is 3x 27.9 + 25x25.5 = 721.2 m. There is no expansion joint in the rail and the ballast on the total length, which is considered as an advantage of this bridge system. The four arches are stiff enough to transmit the big longitudinal braking forces to the ground, without too big deformations of the arches and too much stress increase in the rails [4]

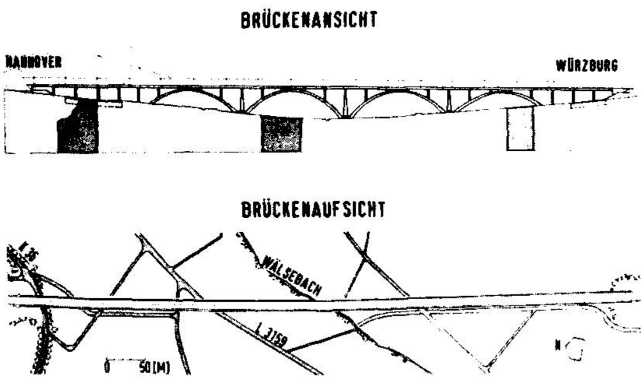


Fig. 6 Elevation and Plan

6. VIADUCT OVER THE RIVER MAIN NEAR NANTENBACH (Fig.2.5)

The New Railroad Line from Hannover to Würzburg is connected to the existing line from Gemünden to Frankfurt by a new 8 km long feeder line, which is under construction from 1990 to 1995. For this line the bridge with the longest span (208 m) of all railroad bridges in Germany, has been designed by Leonhardt, Andrä & Partner. Many different solutions with arches and trusses above the alignment have been studied during preliminary design stage. Finally a steel truss under the railroad has been chosen for tender and execution. The steel structure is in composite action with the concrete deck slab.
More details please find in the report on the Poster Sessions W1.

7. VIADUCT OVER THE RIVER WERRA NEAR HEDEMÜNDEN (Fig.2.5 and Fig.7)

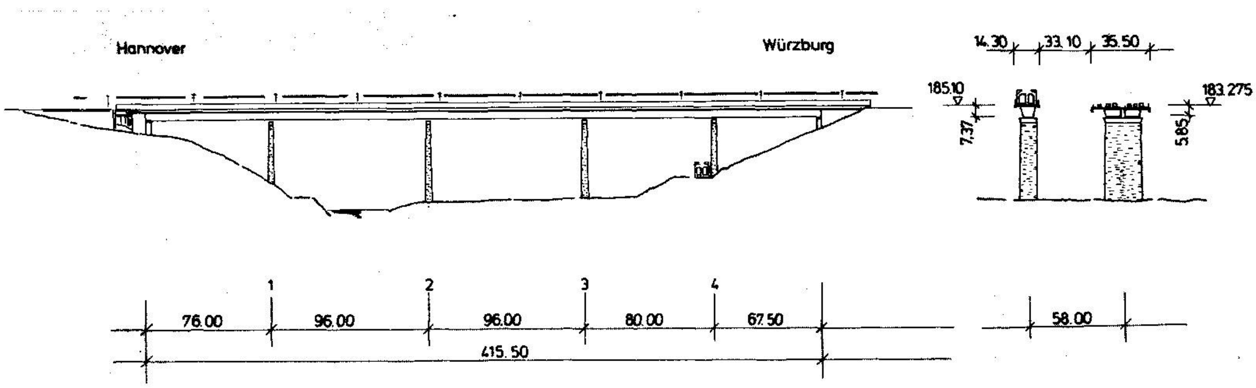


Fig. 7 Werra Viaducts: Elevation and cross-section

This 416 m long bridge for the New Railroad Line crosses the Werra valley 60m above the river. At the same location a similar bridge in length and height was built for the motorway from Kassel to Göttingen 1936/38. The superstructure of the highway bridge had become too narrow and deteriorated. Therefore, it has been decided in 1982 to replace the superstructure of the highway bridge by a new and wider one.

Five consulting engineering groups have been invited for a design competition in order to get the best solution for the complicated task to build a new railroad and a new highway bridge, using the old substructure of the highway bridge and maintaining the highway traffic with 80,000 vehicles per day. This competition was won by Leonhardt, Andrä & Partner with a combined highway and railroad bridge. This design consisted of a steel truss with a wide concrete deck to carry the highway traffic. Steel and concrete had to act together in the so-called composite action. The railroad should have been under the concrete deck between the two planes of the steel truss girder.

Later this bridge has been considered to be too vulnerable and sensitive in cases of accidents, fires or terrorist attacks. Therefore, it was decided to build the highway and railroad bridge separated, parallel in a distance of 50m (Fig.7). New piers had to be built for the railroad bridge in the same distance as of the old highway bridge, i.e. 96m. The new piers were cladded with masonry similar to the old piers of the highway bridge. Such stone works are normally not made nowadays in Germany.

Composite box girders have been used for the superstructure of both bridges. Trusses are aesthetically an excellent solution if only two planes of diagonals have to be made. The solution is not good if there are four planes of diagonals with different distances of the planes and different depth. Prof. F.Leonhardt has been the advisor for the aesthetics of these bridges [5].

8. ACKNOWLEDGEMENT

The New Railroad Lines in Germany cross many wide and deep valleys, requiring big and long span bridges. The Federal Railway Authority was open for new solutions which contribute considerably to the progress of the art to design and build modern long span bridges.

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WORKSHOP 1

Posters

Tests on Steel Railway Bridges over the River Tisza

Essais des ponts ferroviaires métalliques sur la Tisza

Experimentelle Untersuchungen der Eisenbahn-Stahlbrücken über die Tisza

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INTRODUCTION

The crossing of the River Tisza by two continuous 3+4 span heavy-duty, high-speed railway bridges (Fig.1.) offered an excellent chance to carry out delicate investigations relating to the special erection technology and the general behaviour of these type of structures. Some details worth of interest are presented in the poster-contribution of the authors.

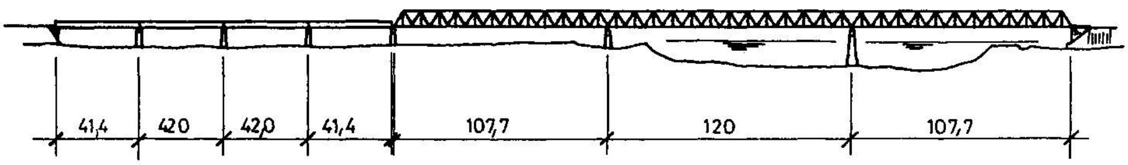

Fig. 1.

The design and arrangement of the steel superstructure were governed by the effort to make the erection operations as simple as possible. The roughly 18 m long plant made welded structural steel units were connected on the site by HSFG bolts.

Both continuous superstructures were pre-assembled on the banks of the river, in the line of the bridge axis. The completely assembled units were then gradually pushed to their place. Temporary trestles were needed only for the truss bridge. Chrom-nickel plated stools were placed atop the supports to distribute the reaction forces during the pushing process and PTFE coated plates were inserted continuously along the slide-track.

THREE-SPAN CONTINUOUS TRUSS BRIDGE

Over the river, the first superstructure is a three span continuous truss bridge with constant heigth of 9000 mm.

Actual measurements reveal the nature of semi-rigid connections of the complex structural system. The obtained normal stresses are confronted to easy to handle calculations of the floor system.

Components of normal stresses near the intersections of diagonals chords of the main are analysed for further refinement of fatigue design. (Fig.2)

Measured deflections show the effect of the rigid connections of bars in the main and the contribution of floor system as well. Special problems, such as effect of lack of fitness are analysed to focus attention to crucial details of bracing system.

Horizontal and vertical natural frequencies, maximum lateral displacements and dynamic coefficients had also been determined and are presented.

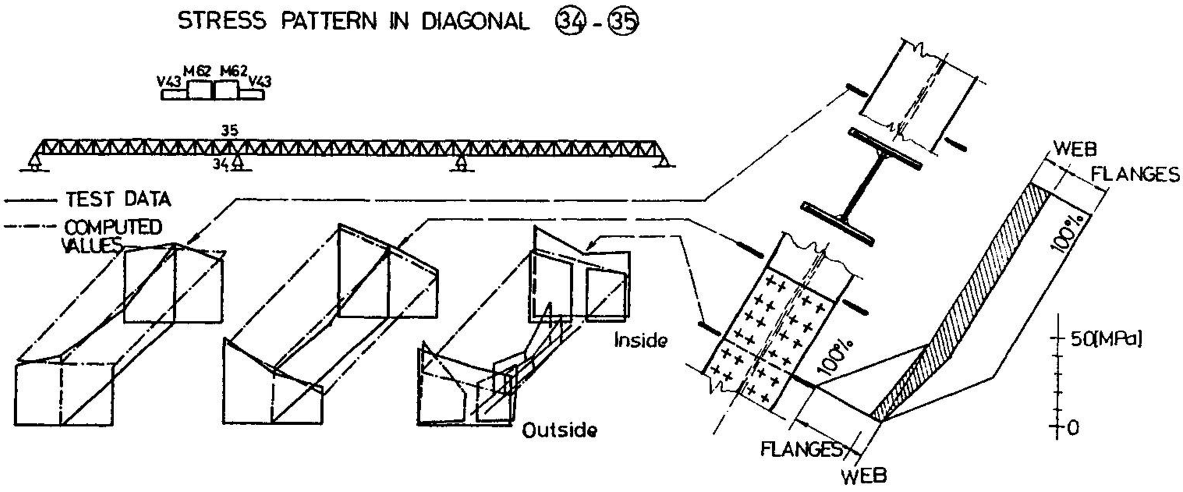

Fig. 2.

FOUR-SPAN CONTINUOUS PLATE GIRDER

The second structure of this bridge system is a four-span continuous plate girder with stringers, cross beams and bracing system similar to the trussed one.

Static measurements showed, that because of the elastic connections among the stringers and main girders, stringers behave as parts of the main girder, but not always with full intensity depending on the position of the loading along the bridge.

The horizontal stiffeners of the web can be taken into consideration totally while during calculation of cross sectional properties of main girders, stringers partially (0,6- 0,8) which are subjected to biaxial bending and warping.

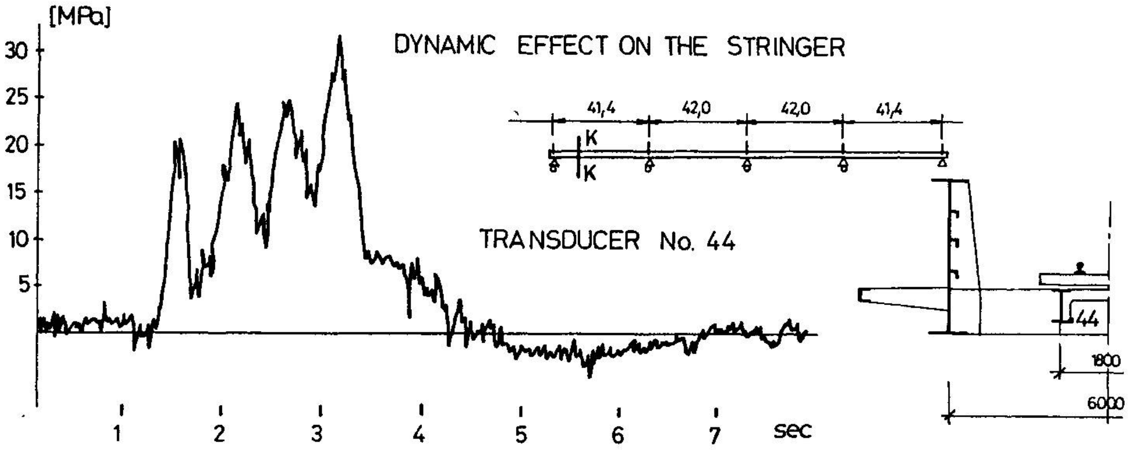

Fig. 3.

Dynamic measurements are illustrated by Fig. 3. Analysis of measured strains on stringers at a train velocity of 60 km/h resulted appr. 15% stress increase because of dynamic effect.

CONCLUSION

Test data prove, that special attention has to be paid to the interaction of different structural components so as to achieve a good agreement at the reality and the results of numerical approaches.

Veitshöchheim-Viaduct: a Concrete Arch Bridge with 162 m Main Span

Viaduc de Veitshöchheim: un pont en arc de 162 m d'ouverture

Talbrücke Veitshöchheim: eine 162 m weit gespannte Bogenbrücke in Beton

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Reiner SAUL
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DESIGN

The New Railroad Line from Hannover to Würzburg crosses the valley of the river Main about 10 km north of Würzburg. The main span (arch) is 162 m. This is the longest span of all concrete railroad bridges in Germany. Over the arch and the piers, a 1262 m long prestressed concrete box girder has been incrementally launched. This has been the longest launch from one end ever done. After launching the continuous girder was made discontinuous at piers 5, 10, 14 and 17, thus dividing the girder into five parts. The reason for this separation is to provide the possibility to replace the superstructure in short pieces of 237.o; 369.5; 214.o; 160.5 or 299.o m, should this ever be necessary. It is assumed that the not prestressed piers and the solid arch never has to be replaced.

Over piers No.5, 10 and 17 is a so called "longitudinal force coupler" and over pier 14 is an expansion joint. The bridge has two fixed points where longitudinal forces (e.g. braking forces) may be transmitted to ground. This is
a) at the crest of the arch, where all longitudinal forces acting onto the superstructure from axis 0 to 14 and
b) at piers 16, 17, 18, where all longitudinal forces acting onto the superstructure from axis 14 to 23

are transmitted to the soil. Rail expansion joints are in the axes 0, 14 and 23. The span between axes 22 and 23 consists of a solid slab 1.4o m deep and built on a scaffolding after the main bridge was launched.

CONSTRUCTION

The arch was built by free cantilevering with the aid of auxiliary cables, which are anchored in auxiliary towers, built of precast concrete segments. The arch is solid and 1.8o to 1.5o m (at crest) thick. Its fresh concrete was chilled from 28° to 10°C by liquified nitrogen. In consequence of the chilling the strength of the concrete could be increased from 55 to 65 N/mm^2. The reinforcement of the arch consisted of not prestressed Dywidag bars with a rolled thread. When these bars met in the closure joint in the crest of the arch, the ribs of the thread did not fit together. The free ends of the bars had to be elasticly twisted in order to be able to turn the coupler nut from one end of a bar onto the other bar. There was not sufficient space to join the two ends of bars by overlapping.

The incremental launching of the heavy concrete superstructure over the slender arch has been an engineering challenge. An arch of 162 m span and a rise of 25 m only, is sensitive, if a beam, three times as deep as the arch, is launched from one side over the arch. The arch was supported by additional cables on the heavily loaded side and it was ballasted by hanged up concrete blocks on the other side.

ACKNOWLEDGEMENT

The preliminary design, the tender documents, the checking of the detailed design and the permanent site inspection has been done by Leonhardt, Andrä und Partner, Consulting Engineers Ltd., Stuttgart, in close collaboration with the client, the German Federal Railway Administration, Nürnberg Division. The detailed design has been made by Obermeyer, Munich. Concrete specialist advisor was Prof. R.Springenschmid, Technical University, Munich. The bridge has been built by the joint venture of the contractors Strabag (Cologne/Würzburg) and WTB (Walter, Thosti, Boswau; Augsburg/Aschaffenburg).

The bridge is a milestone in the development of railroad bridge design and construction and it is a landmark in the beautiful Main valley. The viaduct was opened for traffic on May 29, 1989.

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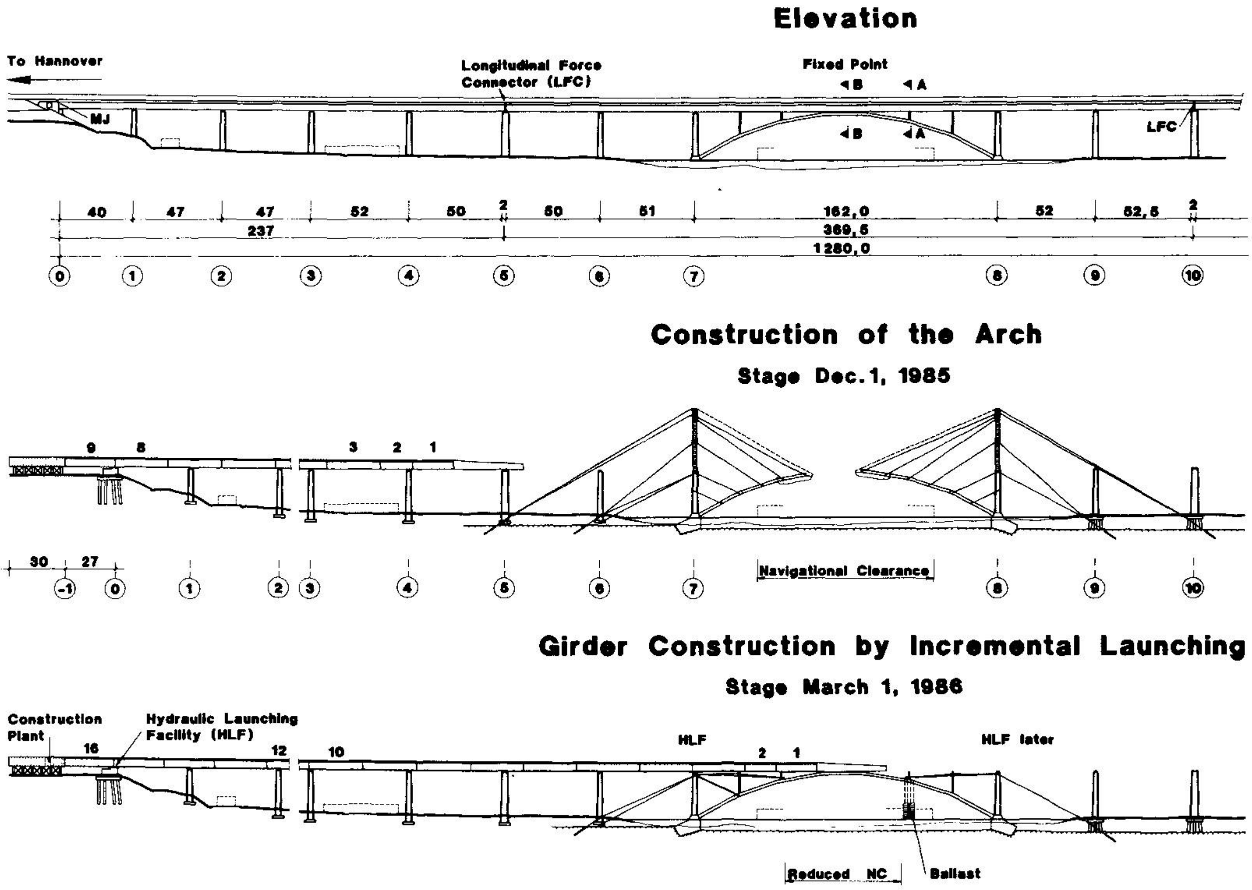

Veitshöchheim-Viaduct

Enz-Viaduct: a 1044 m Long Prestressed Concrete Box Girder Bridge

Viaduc de l'Enz: un pont en poutre-caisson en béton précontraint

Enztalbrücke: eine 1044 m lange Spannbeton-Hohlkastenbrücke

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Reiner SAUL
Senior Supervising Engineer

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DESIGN

The New Railroad Line from Mannheim to Stuttgart crosses the valley of the river Enz about 25 km before Stuttgart. This is the longest viaduct on this new line. It consists of 18 spans of 58 m each, that means a bridge length of 1044 m. The maximum height of the rails above the valley is 47 m.
The bridge is straight on its total length in plan view. The gradient has a change of longitudinal inclination from -0.15 % to +1.2434 %. The radius of the sag-curvature is 60,000 m. This curvature comprises half of the bridge length and is so flat that it can hardly be recognized in the elevation view.
The "General Design" for viaducts of the New Railroad Lines provides for simple supported box girders with a distance of the piers of 44 m. In order not to impair the free view through the scenic valley, the distance between piers has been chosen with 58 m. Additional aesthetical advantages are obtained from the choice of a continuous girder. With a depth of 4.75 m only (compared to 5.3o m required for a simple supported girder) the structure can be designed more slender (piers 3.o m wide instead of 3.5 m).

The total length of the box girder of 1044 m is devided into three equal parts of 348 m each (6 spans of 58 m). Should it ever be necessary, the bridge can be replaced in parts of 348 m in length. A new 348 m long bridge would be built on auxiliary piers aside of the old bridge. By lateral shifting, the old bridge could be replaced by the new one within a few days. The three parts are connected by longitudinal couplers at the piers No.6 and 12 (see poster).
The horizontal forces from braking and accelerating of the trains are transmitted to the abutments (axis 0 and 18). There, the forces are carried onto the ground via hydraulic dampers. The dampers follow slow changes in the width of the rail expansion joints from temperature changes. The dampers react as stiff members in case of the occurance of sudden forces. The bridge is fixed to the piers Nos.7 through 11. This group of five piers holds the bridge in position even if braking should occur many times to the same direction. The group of fixed piers is also able to take the whole braking force, should the hydraulic dampers ever fail.

CONSTRUCTION

The bridge has been built by the incremental launching method on its total length of 1044 m, straight, also in elevation view. Half of its length has been bent into the radius of 60,000 m during launching. Additional prestressing has been provided for these additional moments of constraint.
During launching the three continuous girders have been fixed together to one continuous box girder. After launching the bending stiffness has been released above piers Nos.6 and 12.

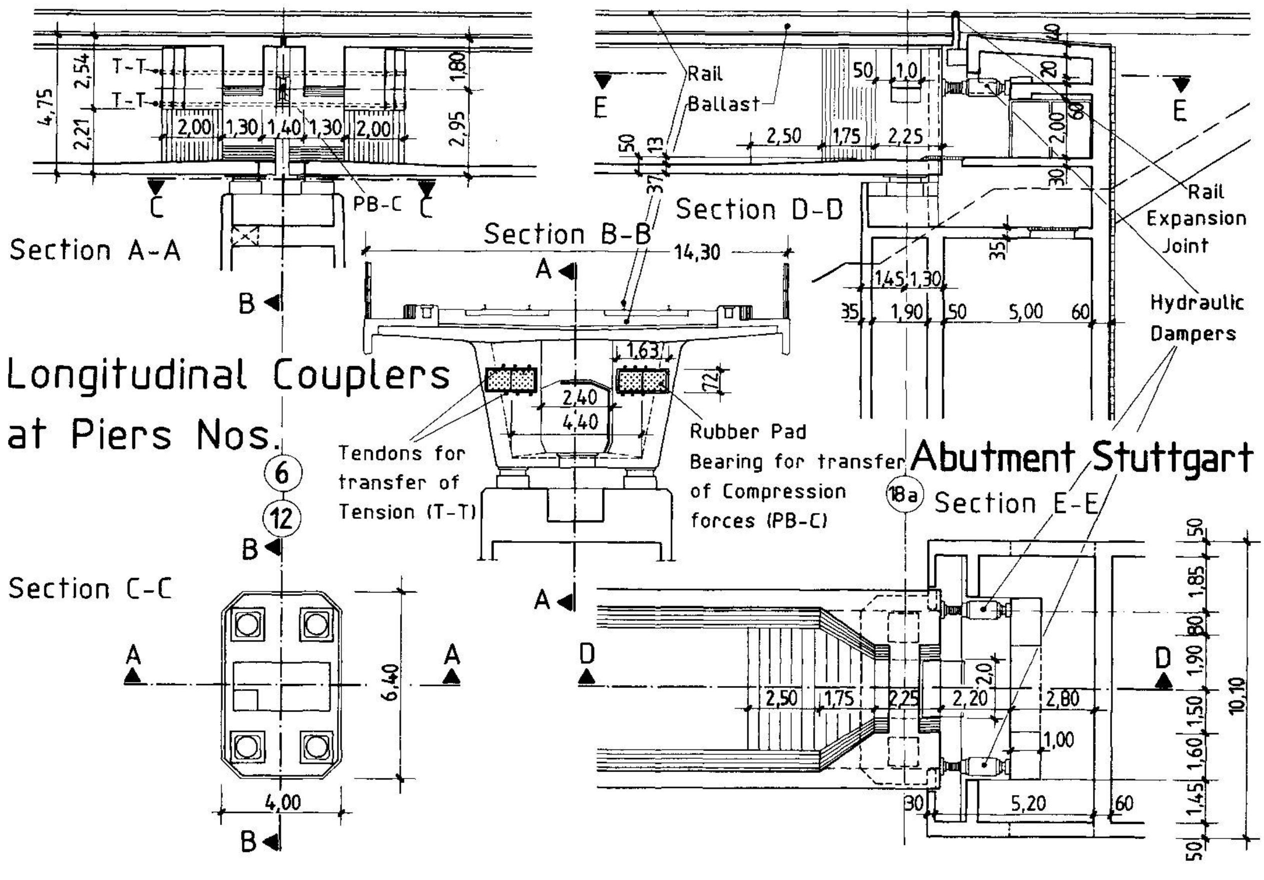
The envelopes of the bending moments show maximum moments near the front end of the girder during launching (see poster). The auxiliary steel launching nose has been 36 m long. The length of a casting element was 29 m, that means half a span length. The trough has been cast on Wednesday, the deck slab on Friday, prestressing and launching has been done on Monday, for a mid span element. The pier elements required two weeks of construction time.
The fabrication plant behind the abutment consisted of three parts of 29 m length each:

a) preassembly of reinforcement and tendons,
b) casting chamber with movable roof and
c) curing chamber with heating in winter.

The concrete cover has been measured by magnetic devices (Proceq Profometer) immediately after removal of formwork. About 40,000 measurements have been made. Also cracks in the concrete have been carefully searched. Improvements in the reinforcement (and in its fixing in the formwork) have been made as a result of the careful inspection during construction works.

ACKNOWLEDGEMENT

The preliminary and detailed design, the preparation of tender documents and the permanent site inspection has been done by Leonhardt, Andrä und Partner, Consulting Engineers, Stuttgart in close collaboration with the client, The German Federal Railway Administration, Karlsruhe Division. The soil and foundation specialist was Dr.-Ing. Christow, Karlsruhe. The checking of the design and the inspection of the prestressing works has been done by Dr.-Ing.Kiefer, Darmstadt. The supervising soil expert was the soil institute of Prof.Smoltczyck and Partners, Stuttgart. The bridge has been built by the joint venture of the contractors Dyckerhoff & Widmann (Munich/Stuttgart), Stumpf (Bruchsal) and C.Baresel (Stuttgart). Due to the very cooperative and open-minded client a most progressive railroad bridge of high quality and durability could be realized. The bridge was opened for traffic on June 2, 1991.



Enz-Viaduct: Coupling devices, hydraulic dampers and rail expansion joint

Railway Bridge with Double Composite Action across River Main

Pont-rail en structure mixte sur le Main

Eisenbahnbrücke mit Doppelverbund über den Main

Reiner SAUL
Senior Supervising Engineer

Wilhelm ZELLNER
Managing Director

Leonhardt, Andrä & Partner GmbH
Stuttgart, FRG

DESIGN

The double track railway bridge across the River Main at Nantenbach will link the new highspeed railway line Hannover-Würzburg to the existing trunk line Würzburg-Aschaffenburg. Due to local conditions, the bridge has a main span of 208 m, a slope of 12.5 ‰ and a radius of 2650 m.

Based on the investigation of numerous alternates, a continuous truss girder with spans of 83.2 - 208 - 83.2 = 374.4 m was found to be the best solution from economic, ecologic and aesthetic point of view, Fig.1, 2. The construction depth varies between 8.5 m at ℄ and the abutments and 16.5 m at the main piers, corresponding to slenderness ratios of 1:24 and 1:13, respectively.

The cross-section, s.Fig.3, consists of
- the truss girders in a mutual distance of 6.o m and with a spacing of 10.4 m;
- the top slab of reinforced concrete, which corresponds to the »Rahmenplanung für Talbrücken« (Masterplanning for valley bridges);
- the bottom chord, of steel in the center of the mid span and of concrete at the piers and in the side spans. The concrete bottom chord limits economically the deformations and makes that fatigue considerations do not govern the dimensioning.

The steel weight is 3300 t or 620 kg/m^2.

CONSTRUCTION

The construction started in early 1991 and is scheduled to be finished in late 1993. The site spans will be erected on auxiliary piers. After pouring of the bottom chord concrete, the center part of the main span, with a length of 120 m and a weight of about 1100 t, will be floated in and lifted. After closure of the center joint, the top slab is poured from ℄ towards the abutments.

ACKNOWLEDGEMENT

Owner is the German Federal Railway Administration, Nürnberg Division (Deutsche Bundesbahn, Direktion Nürnberg). The design and the tender documents were prepared by Leonhardt, Andrä & Partner GmbH, Stuttgart, Germany. The construction consortium is formed by Noell, DSD and Buyck for the steel structure; Strabag and Hochtief for the concrete slabs; and Max Streicher for the foundations and approach bridges.

Fig. 1: General Layout

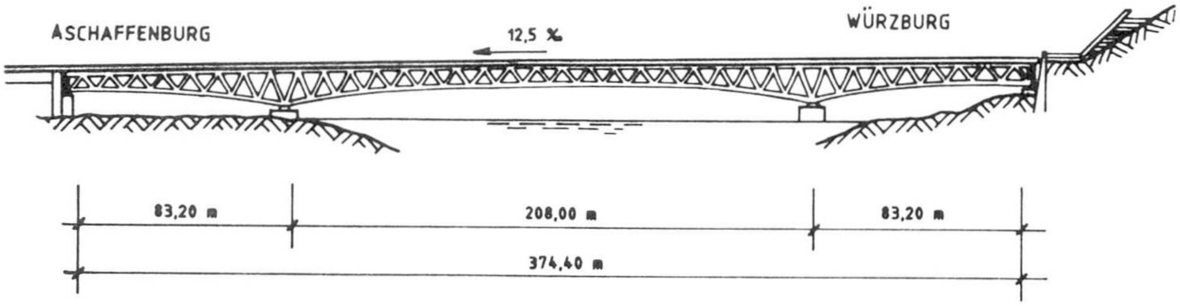
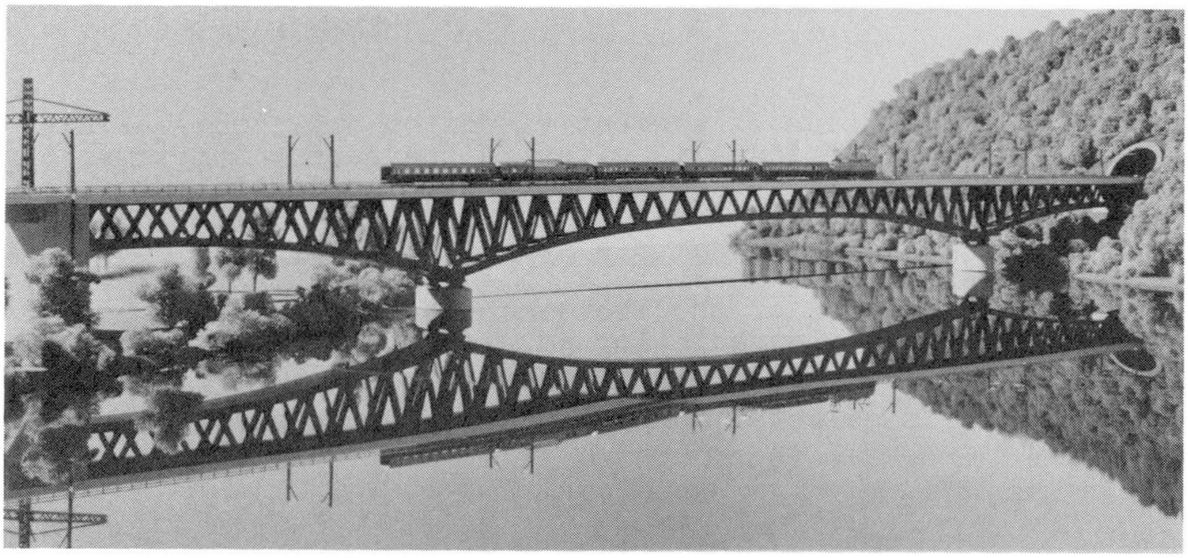


Fig. 2: Foto of model



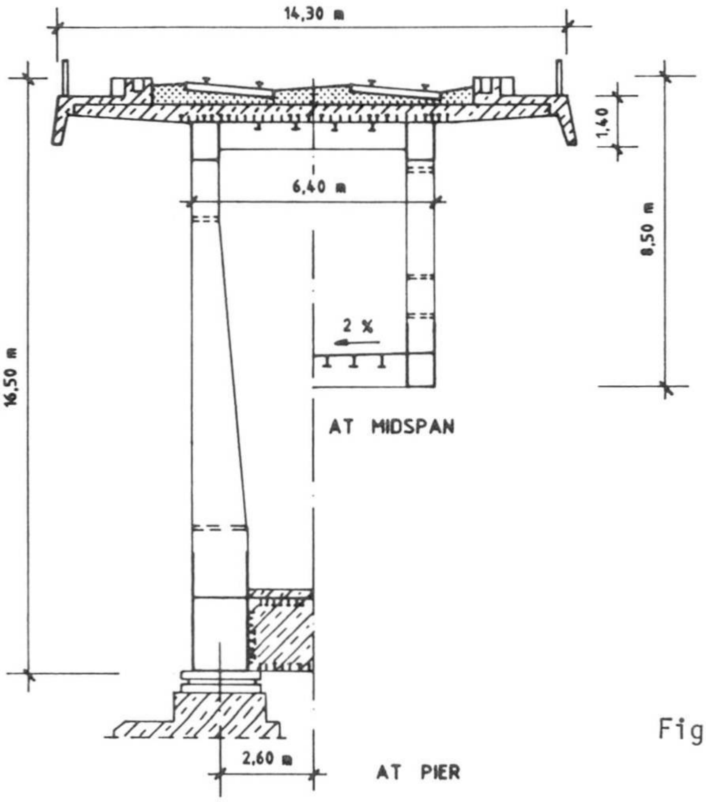


Fig. 3: Cross-section at ℄

Vibration Amplitudes of Steel Railway Bridges

Amplitude des vibrations de ponts ferroviaires métalliques

Schwingungsamplituden in Eisenbahn-Stahlbrücken

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A method of calculation of element vibration amplitudes of high-speed railway metal bridges is given in the paper. Sometimes high-frequency stresses, caused by vibrations, can significantly reduce the durability of metal bridge elements [1]. The operation of such bridge structures proves that vibrations of separate elements of lattice span structures increase with the growth of the trains' speeds. The main cause of the above-mentioned phenomenon seems to be the high-frequency dynamic forces, resulting from the "carriage wheel-bridge railway track" interaction.

Structural models selected for the problem in question are given in Fig.1 and 2.

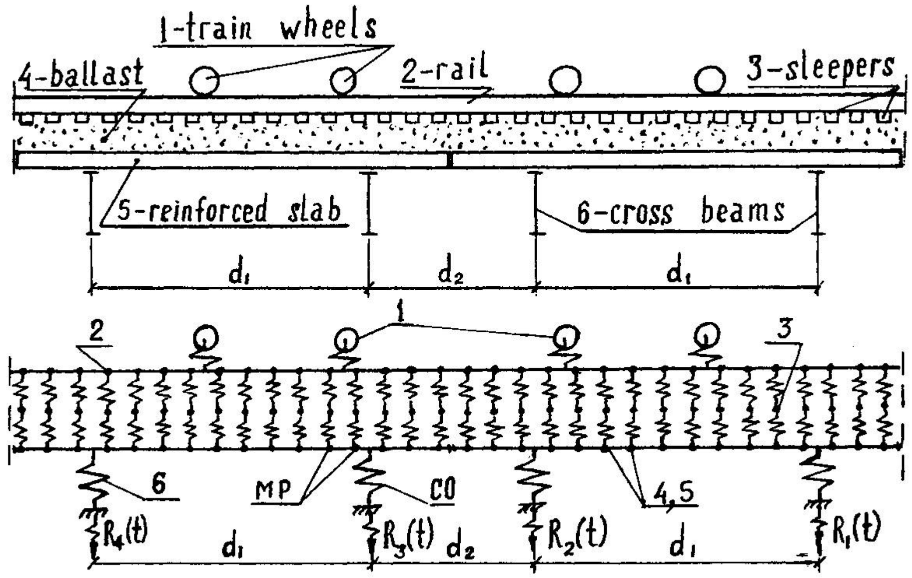

Fig.1

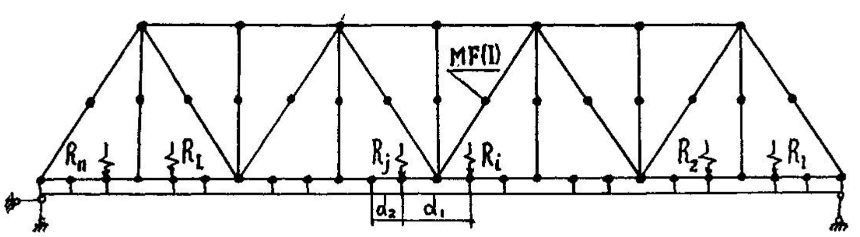

Fig.2

Metal span structures for the high-speed railway bridges are supposed to have rigid bottom chord and the continuous welded track placed on the ballast. The problem is solved in two stages. At the first stage on the basis of both the adopted structural model (see Fig.1), and mathematical model of "carriage wheels - bridge floor - bridge roadway" system variable reactions $R_i(t)$ at the elastic supports with CO stiffness are estimated. These elastic supports model the cross-beams of the span structure. At the second stage accelerations $\ddot{YF}(I)$ and displacements YF (I) of masses which model the elements of the main trusses are determined (see Fig.2). Oscillations of these masses are caused by dynamic forces $R_i(t)$ and are described by the equations as

$$MF(I) \cdot \ddot{YF}(I) = R(I,t) - \Sigma K(I, J) \cdot YF(J) - \Sigma \dot{YF}(J) \cdot B(I,J), \qquad (1)$$

where MF(I), YF(I) are mass and displacement of the first unit of the truss; K(I,J), B(I,J) are coefficients of rigidity and resistance of the system.

$R_i(t)$ - force, transferred to the first mass of the bottom chord of a truss from the bridge roadway, is determined by the following expression

$$R(I,t) = MP(YP(J) + YP(J+1))/2, \qquad (2)$$

where YP(J) and YP(J+1) are accelerations of the roadway slab masses MP, between which the elastic support CO is placed (see Fig.1).

The estimation of YP(J) and YF(J) values is based on the numerical integration of the movement of masses, shown in Fig.1 and 2, by Predicator-Corrector Method [2]. The problem statement allows to take into account one-sided ties at the "wheel-rail" contact and during the "sleeper-ballast" interaction. Integration step T depends on dynamic parameters of the system in question. Generally T is adopted by an order of magnitude less than the minimum period of natural oscillations of the system ($T = 10^{-3} - 10^{-5}$s.)

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2) VASSILYEV B. Numerical Simulation by the Particles Method in the Problems of Mechanics of the Media Deformed. Moscow, Transport,1990, pp 188-197.