SOUS-THÈME 4.2

Structures for Hazard Protection and Composite Structures

Structures pour la protection contre les phénomènes accidentels et structures mixtes

Bauwerke für den Schutz von Unfallphenomenen, und Verbundbauwerke

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Concrete Structures as a Safe Engineering Response to Environmental Catastrophes

Structures en béton : réponse technologique sûre aux catastrophes écologiques

Betonstrukturen als sicherheitstechnische Antwort auf Umweltkatastrophen

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SUMMARY
Innovations must not be limited to the classical areas of concrete construction. As a matter of fact, they are needed in numerous new fields. One of these is the protection of the environment against catastrophes caused by technological failures. The article outlines the concept of a protective technology and of new concrete structures to guarantee a safe enclosure of raw materials, products and waste materials in an industrial cycle. The objective to be achieved is to strictly limit any damage caused by accidents.

RÉSUMÉ
Les innovations ne doivent pas se limiter aux domaines classiques d'utilisation du béton. Le besoin s'en fait sentir dans nombre de domaines nouveaux, en particulier celui de la protection de l'environnement contre les catastrophes d'origine technique. L'article expose les principes de base d'une technique de protection. Il indique selon quels principes les nouvelles structures en béton doivent être conçues en vue de constituer pour les matières premières, les produits ou les déchets impliqués dans le circuit industriel une enceinte offrant toutes garanties de sécurité. L'objectif visé est, quelle que soit la probabilité des accidents, de limiter strictement les dommages qui leur sont dûs.

ZUSAMMENFASSUNG
Innovationen dürfen sich nicht auf die klassischen Arbeitsgebiete des Betonbaus beschränken. Gefragt sind sie vielmehr auf zahlreichen neuen Gebieten. Dazu zählt der Schutz der Umwelt vor technikverursachten Katastrophen. Skizziert werden das Konzept einer Schutztechnik und neue Betonstrukturen zur Gewährleistung der sicheren Umschließung von Rohstoffen, Produkten und Abfallstoffen im industriellen Kreislauf. Ziel dabei ist es, unabhängig von der Wahrscheinlichkeit von Unfällen, den mit Unfällen verbundenen Schaden streng zu begrenzen.

1. INTRODUCTION

Modern industrial society has been thrown into a state of extreme uncertainty by spectacular cases of technological failures with catastrophic consequences. Names such as Seveso, Mexico City, Bhopal or Schweizerhalle - and last but not least Chernobyl - may be regarded as typical of this uncertainty. The consequence is that society welcomes a product but at the same time rejects the production process it involves. The public continues to expect general affluence and general safety. Affluence and safety are also political aims, as indeed they surely must be, but they can only be realized - if at all - by the continuing and increasing use of large-scale industrial technology. Any technology however has potential risks that may result in damage - indeed must do so at some time or other. Only a technology that we decide to do without it is devoid of any risks.

But even when the damage potential is catastrophically high, accidents need not necessarily result in disasters. This report aims to demonstrate this for a number of non-nuclear technologies and at the same time will attempt to answer the question: "How safe is safe enough?" It is based on development work carried out by Dyckerhoff & Widmann AG. Limitations of space do not allow a description of individual projects and the solutions found, but instead we will outline a preventive engineering concept for the entire industrial cycle: raw material, product, waste material, in which all these solutions are integrated and from which they were developed. What is technically feasible, and what is economically acceptable - both questions are dealt with here.

Applications of this kind for concrete construction are extremely new and constitute both an opportunity and a challenge.

2. MEXICO CITY AND BHOPAL - TWO CATASTROPHES

Most events with catastrophic consequences outside what may be termed "the industrial fence" - and this is what we mean by environmental disasters - have their origin in technical disturbances "inside the fence" which have got out of control.

In Mexico City, after a leakage in a PEMEX liquid gas plant, several explosions and devastating fires occurred, destroying the plant almost completely and devastating the surrounding housing estates. According to official figures more than 500 people were killed, with more than 7,000 seriously injured. Material damage is put at several hundred million US $. The area affected was 2.5 km in diameter.

In Bhopal, India, approximately 25 tons of highly toxic methylisocyanate were released from one of several storage tanks in a chemical factory belonging to Union Carbide. In this disaster about 3,000 people who came into contact with this toxic substance were killed, and about 200,000 suffered injuries to health. The amount of subsequent damage is incalculable. Lethal levels of poisonous gas were exceeded over an area of 40 km², with a diameter of approximately 7 km. India demanded more than US $ 3 billion compensation from Union Carbide for damage to property, health and environment.

3. SAFETY AIMS AND SAFETY MEASURES

Catastrophes are "infrequent events" - cases of technical failure with extreme consequences, as shown by the disasters of Mexico City and Bhopal, but the probability of their occurrence is really small. In the risk spectrum these are represented by the dark curves (Figure 1). A risk is usually defined as the product of the probability of occurrence and the potential amount of damage. Below a limit delineated by the relevant practical experience all indications of probability have a purely hypothetical character. For facilities involving a high level of risk - these are the ones we are dealing with - there is a gap between the reliability factor obtained by calculations and that dependent on operational experience. This gap can never be closed, however exact our calculations. Chernobyl has once again brought home this fact.

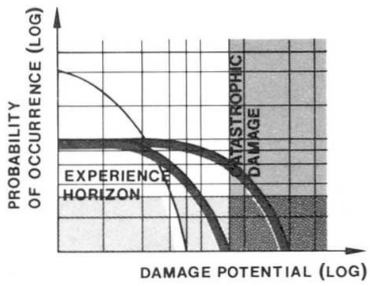

Fig. 1. Risk spectrum

The necessary conclusion to be drawn from this is that a safety aim that only attempts to progressively reduce the probability of occurrence of "infrequent events" is insufficient. What is needed additionally is the restriction of the amount of damage to an acceptable, no longer catastrophic level. This has to determine our entire course of action. Only when we demand that the dark area representing catastrophic damage in the risk spectrum should also be avoided will our safety aims be comprehensive enough.

After Bhopal Union Carbide equipped several plants with a computer-controlled safety system connected to a meteorological station. Based on the dispersion values of any escaped chemical and the weather conditions it aimed at the rapid calculation of the direction and speed of a toxic cloud. Thus, at least in theory, makes it possible for a catastrophe alarm warning to be given in good time and for adjoining housing to be evacuated. This is undoubtedly one way of restricting the damage inherent in any given risk. Active safety systems such as this, however, require an extremely high standard of quality and high redundancy levels, but even then they are still not safe from "computer bugs".

Both active and other additional, i.e. passive safety systems are aiming at a technology that can fail without catastrophic damage. Only a technology safe within such limits is acceptable in the long term.

4. CONCRETE COMPONENTS AS PASSIVE SAFETY SYSTEMS

The aim in a safety context of this kind is the introduction of a passive safety system which, independent of the probability of accident consequences, rigorously restricts the volume of damage caused by such events. This can be achieved by structural safety measures using concrete components. What makes this passive safety systems so valuable is their almost deterministic reliability when compared to active safety systems. They are independent of accidents caused by human error, which for 80 % of all accidents is given either as the only cause or together with technical failure. A typical example of passive safety is the well-known firewall, which prevents fire from spreading not by starting any process or by automatic control technology, but simply by its sheer presence.

The facilities in Mexico City and Bhopal had nothing of this sort. In Chernobyl too safety relied to a great extent on "active safety" principles.

5. THE AIM OF SAFETY MEASURES: ENSURING ENCLOSURE

In all safety measures the main aim is nothing else but to ensure that hazardous materials are enclosed against external and internal influences over relatively long periods.

6. BURST-PROOF ENCLOSURE SYSTEMS

A small leakage in Mexico City caused a disaster because steel pressure vessels were able to explode like bombs. Explosions of this kind are termed "BLEVEs", which stands for "Boiling Liquid Expanding Vapour Explosion". A BLEVE is a physical explosion of the vessel, which "bursts" and is the centre of a more or less devastating wave of pressure, and - if the contents are flammable - of a wave of heat as well. The result is then a fireball. The waves of pressure and heat together with container fragments acting as missiles may also cause adjacent containers to undergo a BLEVE. Any facility can be completely destroyed by such a "domino effect", as happened in Mexico City.

Similar events can and will happen again elsewhere, because only very few containers today are likely to be sufficiently safe against BLEVEs.

What are needed are pressure vessels with a structure that is not only unaffected by BLEVEs but is also as far as humanly possible BLEVE-proof.

These containers are now available and can be built. Made of reinforced and prestressed concrete of varying shapes, they form an entire family of containers: for cryogenic and non-cryogenic liquid gases, and for flammable or toxic liquids [1].

All containers in this family are:
- safe against the effects of catastrophes, as far as humanly possible

and
- technically and economically efficient.

When operating pressures are high the container is in the shape of a spherical shell (Figure 2). The dimensions in Figure 2c relate to pressure liquefied propane. Figure 2d represents the construction process in which the sphere, consisting of several parts, is expanded and thus prestressed, then being closed to form a monolithic shell.

In contrast to Mexico City the containers in Bhopal are all very small and cylindrical. A concrete enclosing shell in Bhopal could also be cylindrical in shape. The ideal solution would be to combine them into larger units again spherical in shape.

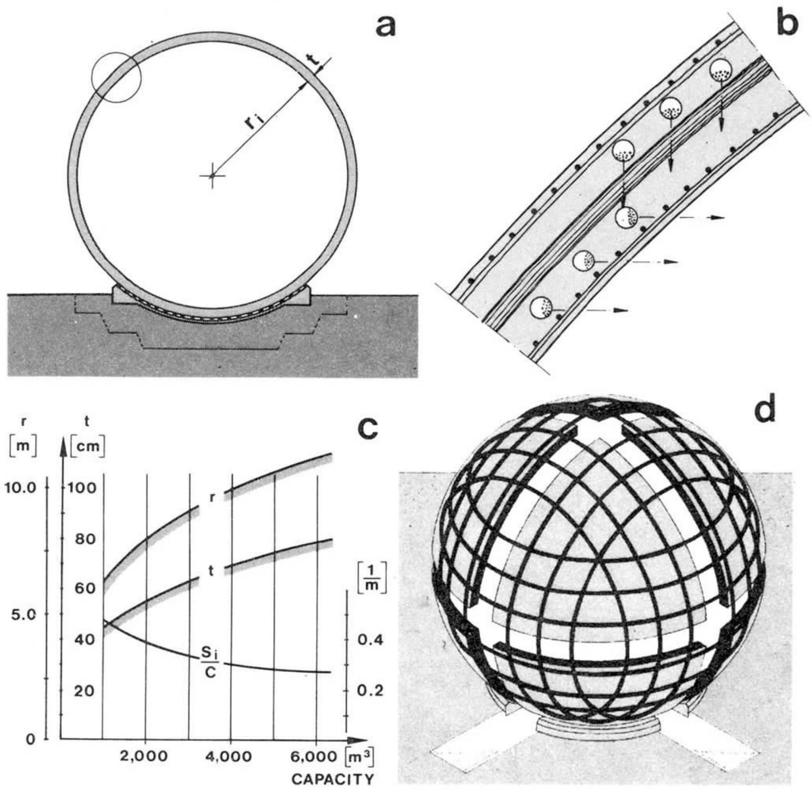

Fig. 2. Spherical concrete pressure vessel

Concrete pressure vessels, owing to their structure, have high redundancy levels. They cannot burst and are resistant to perforation and fire, e.g. [1]. In steel pressure vessels, on the other hand, the safety elements form a tandem connection, which is why once failure begins the process cannot be stopped. This was proved - not for the first time - by Mexico City.

Safety is expensive and more safety is even more expensive. We are used to this and are ready to accept it. The spherical concrete vessel - to restrict ourselves to this shape - should therefore cost more than one made of steel. In fact the reverse is true (Figure 3). The concrete container will cost less. This applies to the entire range of different capacities. Figure 3 shows the results of a cost comparison based on the pressure storage of liquid propane - in other words, one suitable for Mexico City. In such a case concrete containers can cost up to 40 % less than steel containers designed on the basis of the ASME Code. So we have good reason to assume that the new quality of safety we are discussing here and the higher availability connected with it will cost considerably less than what is still being built world-wide today.

The full economic advantage will and can display itself only if an entire facility, including all smaller safety distances and all other safety elements, is designed on the basis of concrete containers.

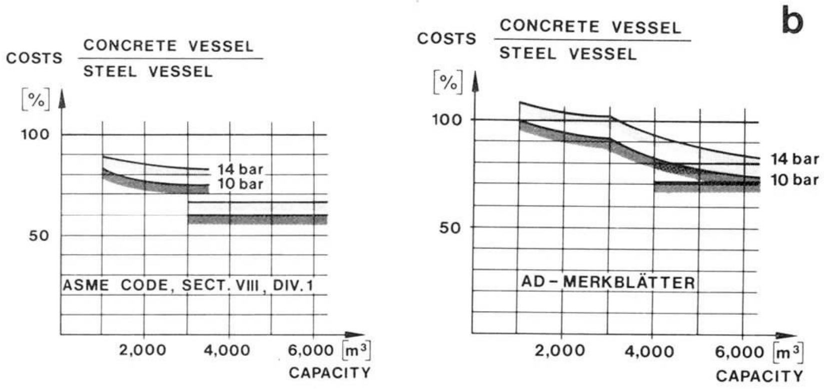

Fig. 3. Costs ratio of spherical pressure vessels

7. "MATERIAL TECHNOLOGY" AND "SYSTEM TECHNOLOGY" IN THEIR SIGNIFICANCE FOR ENCLOSURE SYSTEMS

So far we have discussed the enclosure of environmentally hazardous raw materials and products. However, waste materials are also involved in the material cycle. They too require to be reliably enclosed, frequently over very long periods of time, unless they can be recycled or their eluates are not harmful.

While the composition of raw materials and products - and thus how they will affect the enclosing system - are known, this does not as a rule apply to waste materials. If this is so, then material technology will lose its dominating role in the design of reliable enclosure systems, because we do not know which enclosing material - or whether there is one at all - can resist the effects of the waste material. System technology must now take the place of the material technology as the dominant technology. Only with its help will we be able to counter at the earliest stage any material-induced failure in the enclosing system.

8. SAFETY AIMS AND SAFETY MEASURES FOR MATERIALS WHICH ATTACK THE ENCLOSING SYSTEM IN A NON-DIFFERENTIABLE MANNER

The concept of risk is no longer applicable to material systems that can destroy the system enclosing them, so to speak as a result of their inherent nature. In such cases the risk of failure can no longer be limited. The safety aims detailed in Section 3 above must therefore be extended accordingly. From a structural point of view this means that the enclosing system can be guaranteed safe only if, and only then, it is designed in all its parts to be accessible to checks, to be repairable in a way that is accessible to checks - repeatedly repaired if necessary - and to be renewed. This is a new, entirely unusual design aim.

From this point of view enclosing systems for waste materials must differ markedly from those designed for raw materials and products, with regard to attention paid to safety and thus also to construction. Existing sytems do so - unfortunately in the reverse direction. Many waste storage dumps existing today are preprogrammed to becoming future liabilities, because the reliability of the enclosing systems continues to be - and is continually - wrongly assessed.

Waste material storage facilities for substances representing an environmental risk must be in the form of buildings - there is no way of avoiding this if we are serious about environmental protection.

9. ENCLOSING SYSTEMS THAT CAN BE CHECKED, REPAIRED AND RENEWED

We will describe a number of solutions to problems involving the storage of waste materials, these being based consistently on the safety aim of an enclosing system that can be checked, repaired and renewed [2]. This enclosing system has, as we shall see, a modular construction. This means that all solutions are adaptable, adaptable to very different requirements depending on whether - and how - the waste substances have been pre-treated.

All shapes are possible for the enclosing system, examples being mound-shaped dumps (Figure 4a) and container dumps (Figure 4b).

In the detailed cross-section of a container dump here considered as an example the modular construction can be easily recognized (Figure 5a). The elements from inside to outside are: Plate filter, plate lining, inside wall parts with supports to outside wall. As far as possible filter and lining material will be adapted to suit the task in question - in this example we dealing with the storage of untreated waste. Inside and outside walls are made of concrete.

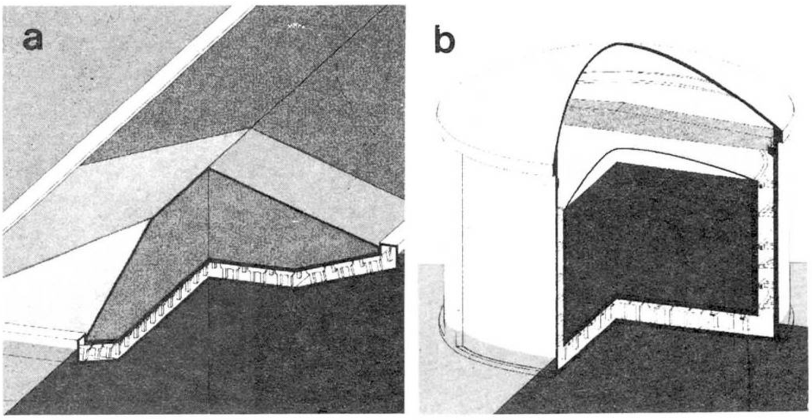

Fig. 4. Controllable, repairable and renewable enclosure systems for wastes

How is repair work to be performed? How are parts to be replaced? This is also shown in the cross-section details given in Figure 5b for the container dump. The working operation is roughly as follows: Compact the waste locally, with the container remaining filled. Separate the lining seams, move the enclosing element hydraulically into the annular space - into the working position - repair and renew defective elements, e.g. plate filters, plate linings or even the concrete of the enclosing elements, return the element into its normal position, weld the seams of the lining together again - and the entire operation is completed. In the base area of the enclosing system work is performed in a similar manner. The same applies to the mound-shaped-dump.

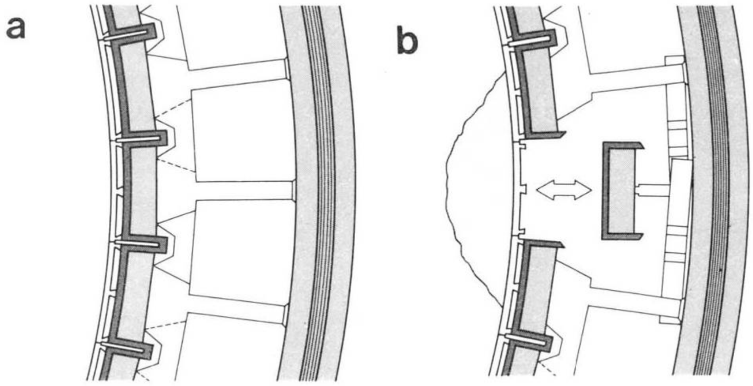

Fig. 5. Container-dump. Details of cross-section

This enclosing system is quite consciously based primarily on the principle of passive safety.

The design of the system allows for errors. The system is quite intentionally designed to accommodate a phase of trial and error. This means that technical and human errors can be kept under control and corrected.

The design however also contains redundancy elements, with all parts of the system being completely accessible. All working operations - the carrying out of checks, repairs, renewal work, including functionality tests, can be repeated as often as required.

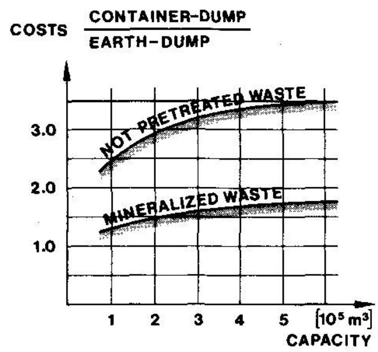

Fig. 6. Costs ratio of waste dumps

Reliability in waste storage enclosing systems does cost more (Figure 6). Since the construction of earth waste dumps with a combined sealing enclosure and temporary roofing is still relatively frequently considered as a solution to the waste dump problem, let us use a dump of this kind as a comparison. The mineral sealing course should have a thickness of 1.5 m and the plastic lining course should be 3 mm thick. For non-pretreated waste - in this case the dump is then a biological and chemical reactor - the container dump costs 2 to 3.5 times as much as for this simple earth dump - for dumps with small capacities twice as much, for those with larger capacities 3.5 times. If the waste has been mineralized costs for the container dump will be halved, those 2 to 3.5 times being reduced to 1 to 1.7 times. Markedly cheaper than the container waste dump is the mound-shaped dump.

These comparisons show that expenditure on reliability does not increase exponentially as is sometimes assumed, but what does rise exponentially is the limitation of damage to the environment. Earth waste dumps with combined sealing enclosures permit neither checks nor repairs, and, since the effects caused by wastes are for the most part non-differentiable, they are with great probability condemned sooner or later to their own "ruin" from within themselves, and to the failure of the enclosing system. Claiming that reliable leakage rates for such a failure can be defined is an illusion.

10. CLOSING REMARKS

Today's trial and error procedures are no longer appropriate when environmental catastrophes caused by technology are concerned. The danger potential is too great for this, and a detrimental interaction of materials and factors which increase uncontrollably through their effect on each other is too probable. We can no longer live with this situation, and we do not have to. This report attempts to show this.

The task facing us all is to bring concrete structures as a preventive answer to environmental catastrophes into the public discussion of safety - a discussion that up to now has not been given a direction and which is stumbling from one disaster to the next. Used in the right way technology can and also must serve to protect the environment, and thus to conserve the quality of life. The better this principle is understood and put into practice, the less society will be tempted to reject technology.

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Design and Construction of Tall Reinforced Concrete Buildings in a Seismic Country

Conception et réalisation de bâtiments de grande hauteur en béton armé
dans un pays sujet aux tremblements de terre

Entwurf und Ausführung von hohen Stahlbetonbauten in einem erdbebengefährdeten Land

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SUMMARY
This paper outlines several innovations in the conception of tall reinforced concrete buildings in Japan. Due to the high seismicity, the structures must be designed and built to withstand large alternating forces and preserve large ductility. Taking the case of a 30-storied residential building as an example, design procedures and safety confirmation methods as well as construction practices are described.

RÉSUMÉ
Cette contribution présente plusieurs innovations concernant les bâtiments de grande hauteur en béton armé au Japon. Ductilité et résistance à de grandes forces alternées doivent être absolument respectées en zone de haute séismicité. Le cas d'un bâtiment résidentiel de 30 étages sert à illustrer la conception, les moyens utilisés pour assurer la sécurité ainsi que les méthodes de la construction proprement dite.

ZUSAMMENFASSUNG
Der Beitrag stellt verschiedene Innovationen in der Konzeption von hohen Stahlbetonbauten in Japan vor. Infolge der grossen Erdbebenbeanspruchung müssen die Bauten grosse wechselnde Beanspruchungen ertragen können und eine grosse Duktilität ausweisen. Anhand eines 30-stöckigen Hochhauses werden die Projektierung, die Ausführung und die Sicherheitsaspekte beschrieben.

1. INTRODUCTION

It had been considered until a decade ago that, due to occurences of frequent severe earthquakes in Japan and brittle behavior of concrete structure in general, the construction of a tall reinforced concrete building was impossible. Learning lessons from many damages by earthquakes such as Tokachioki(1968,Japan) and San Fernando (1971,USA), and driven by economic demands the authors developed a new design and construction method for tall reinforced concrete buildings after solving problems on aseismic capabilities. The method called HiRC was already successfully applied to several tall buildings as shown in the references. [1,2,3]

As of March 1987, the completed reinforced concrete buildings over 50 meters high in Japan are limited to those by HiRC design and construction method. This paper is a general report on the innovative method for design and construction of other 30 storied concrete buildings which are now under construction in the City of Kawasaki near Tokyo.

2. OUTLINE OF THE BUILDING

The building is of 30 stories with the eaves height of 87.2 meters and has 230 housing units. Typical framing plan is approximately 30 meters square with recessed central portions on the four exteriors as shown in Fig.1 - 3. Building structure is composed of moment resisting frames by pure reinforced concrete structurally separated from walls, which makes possible the flexible housing layout at every floor.

In order to resist severe earthquake stresses especially induced in exterior columns on the lower story, following three items are, although unprecedented in Japan, introduced in the design practice.
- to use high strength concrete up to 420kgf/cm. (Concrete up to the 360kgf/cm was used for previous buildings.)
- to provide additional core-reinforcing bars in exterior columns on the lower story.(Prestressing tendons were used in case of previous buildings in order to reduce high tensil stresses)
- to use large subsoil diaphram walls in addition to piles with enlarged base in consideration of potential liquefactions as well as vertical and horizontal loadings.

3. DESIGN PRINCIPLE

Seismic design of the building structure is conducted through two-stage designing. The first one is based on so called allowable stress design method, while the second is on ultimate strength design method. Design earthquake force applied in the former stage is defined by the preliminary earthquake response analysis, against which the stresses induced in beams and columns should be within allowable limits.

The second stage of design is to check the stresses of the column and joining portions on condition that all the beams have already yielded, which implies that strong-column and weak-beam design principle is indispensable and the design procedure should take a form of feedback system. Typical cross sections of columns and beams by the final design are shown in Fig.4. High strength concrete and additional core-reinforcing bars make the unit structural costs reasonable compared with those of the previous lower buildings.

Fig.1 Building under construction

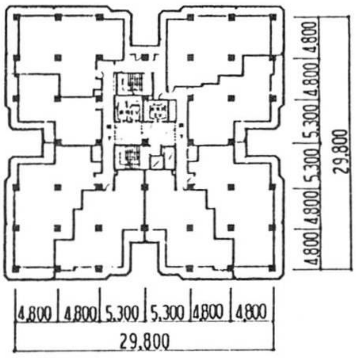


Fig.2 Typical floor plan

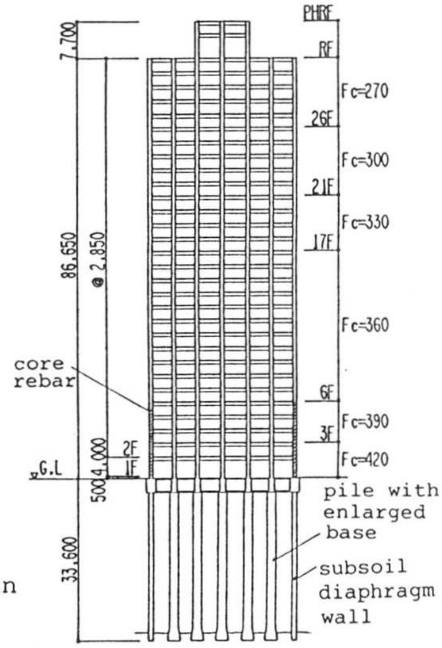


Fig.3 Framing elevation

construction

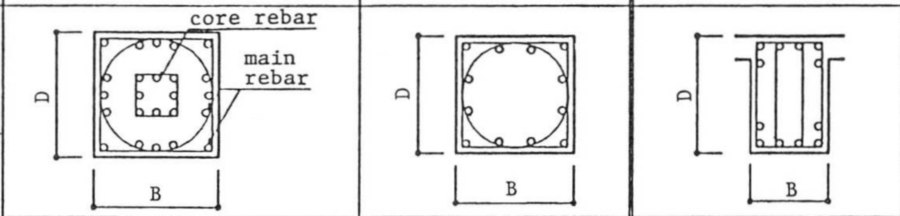
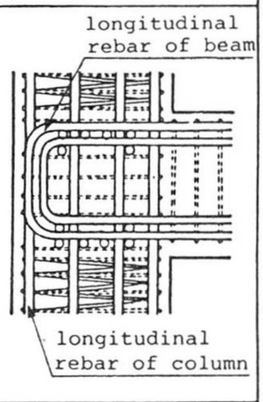
	Exterior column				Interior column			Beam			U-shaped anchorage
Cross section											
floor	B x D	main rebar	core rebar	hoop	B x D	main rebar	hoop	B x D	main rebar	stirrup	
30 ↓ 2	800×800 ↓ 850×850	12-D32 ↓ 16-D41	— ↓ 8-D41	D13 @150 13φ @150 ↓ D16 @100 16φ @100	750×750 ↓ 800×800	12-D32 ↓ 16-D41	D13 @150 13φ @150 ↓ D16 @100 16φ @100	500×750 ↓ 600×1000	4-D25 2-D22 ↓ 4-D41 2-D38	4-D13@175 ↓ 4-D16@125	

Fig.4 Typical cross sections

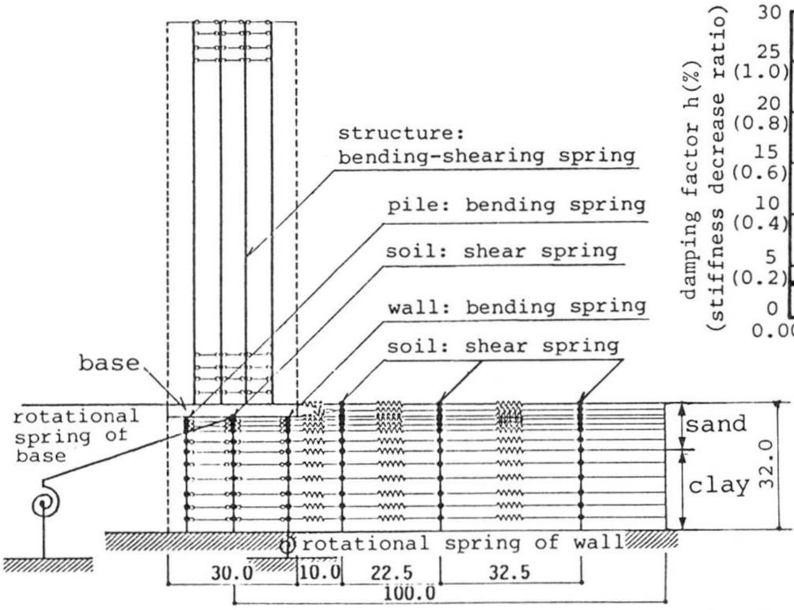


Fig.5 Modelling in earthquake response analysis

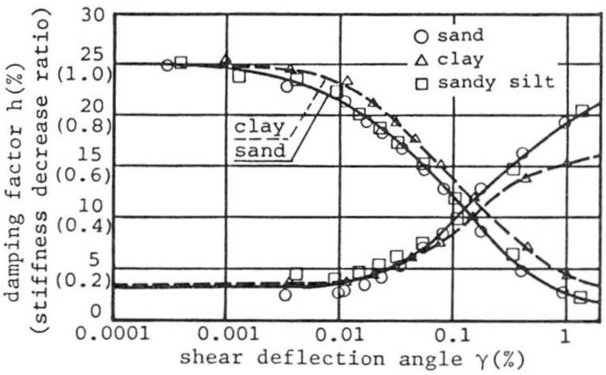


Fig.6 Dynamic characteristics of sub-soil

4. SAFETY CONFIRMATION

Safeness of this building against earthquakes are confirmed by the response analysis considering soil-pile-structure interaction as well as by the structural test of the component members.

4.1 EARTHQUAKE RESPONSE ANALYSIS

The vibration model used in the earthquake response analysis is shown in Fig.5, where bending-shearing springs equivalent to superstructure's framings are linked with each other using rigid floor assumption while shearing springs representing subsoil, are also connected by axial soil springs. Stiffness and damping factor of these subsoil elements are evaluated as shown in Fig.6 from experimental data using triaxial testing on boring samples from the construction site. Subsoil diaphragm and piles are also taken into account in the modelling, where rotational and horizontal movements of the foundation are defined by the bending and axial movements of these substructures.

The input earthquake wave is the EW component of the SENDAI TH038-1 which is one of the severest earthquake records in Japan. Intensity of the motion is defined to be 250 gals and 400 gals in maximum accelerations at the ground surface.

By the eigenvalue analysis on this model, it is recognized that the fundamental vibration periods are 1.85 second and 0.75 second which correspond to those of building and subsoil respectively. From the results of nonlinear response analysis in case of the earthquake with maximum acceleration of 250 gal, maximum responses are shown in Fig.7 and 8 as examples in the form of envelopes of shearing force and overturning moment that occured on each mass point. Stresses of members induced by these dynamic forces are recognized to be within allowable limits.

Nonlinear dynamic response analysis against the earthquake with maximum intensity of 400 gals is also conducted and the sufficient safeness of the designed building structure is confirmed in view of both strength and ductility.

4.2 Structural Test

Although previous testings on more than 100 specimens of structural members confirmed the appropriateness of design method [4], columns with additional core-reinforcing bars newly introduced in this building should be also confirmed using high strength concrete. Two specimens representing exterior columns on 2nd story are approximately 1/3 scale of actual members as shown in Fig.9. and are subjected to both horizontal and vertical earthquake forces after the axial fixed loading. No.1 specimen is loaded until final horizontal distortion under tensile field, while No.2 under compressive one.

Result of test shown in Fig.10 clarifies that the tensile column preserves its horizontal strength up to 2.5 times of the design shear stress with the deformation angle of 0.04 radian. Compressive column shown in Fig.11 reaches its horizontal ultimate strength under the deformation angle of 0.01 radian and also preserve its vertical load carrying capacity even until the deformation of 0.02 radian, which both prove sufficient abilities for an aseismic structural member.

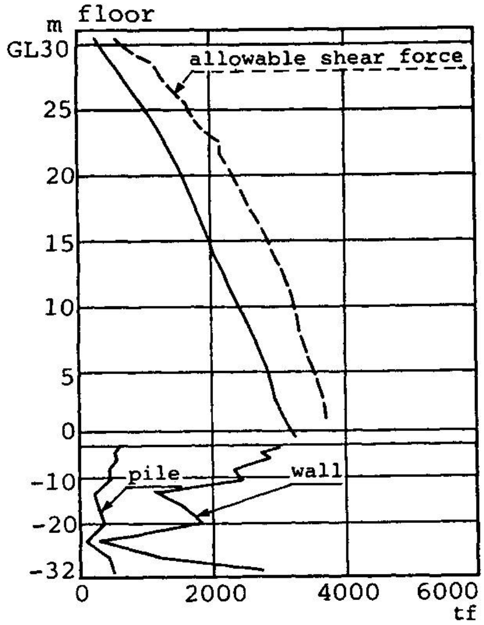


Fig.7 Maximum response shear force

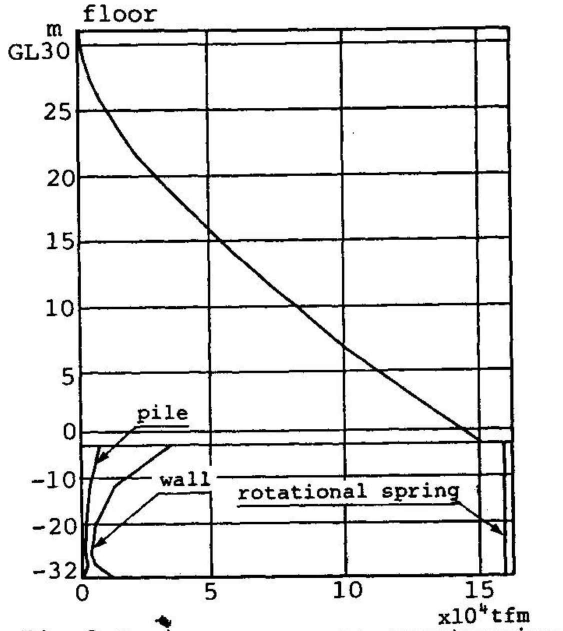


Fig.8 Maximum response overturning moment and bending moment

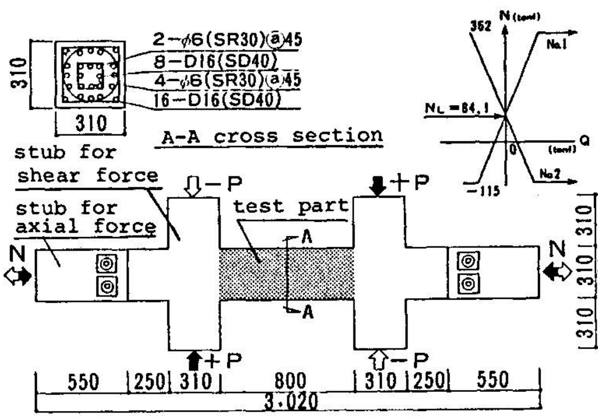


Fig.9 Specimen and loading pattern

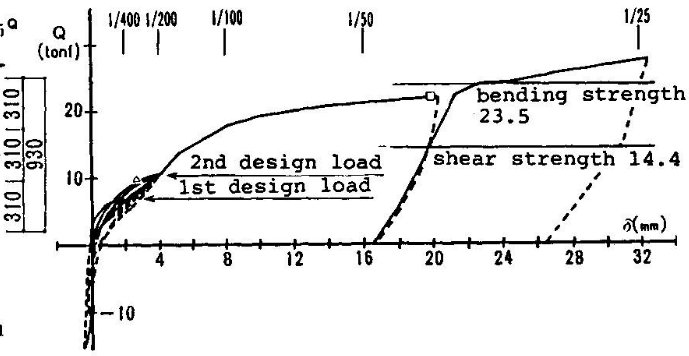


Fig.10 Load-deflection curve of No.1

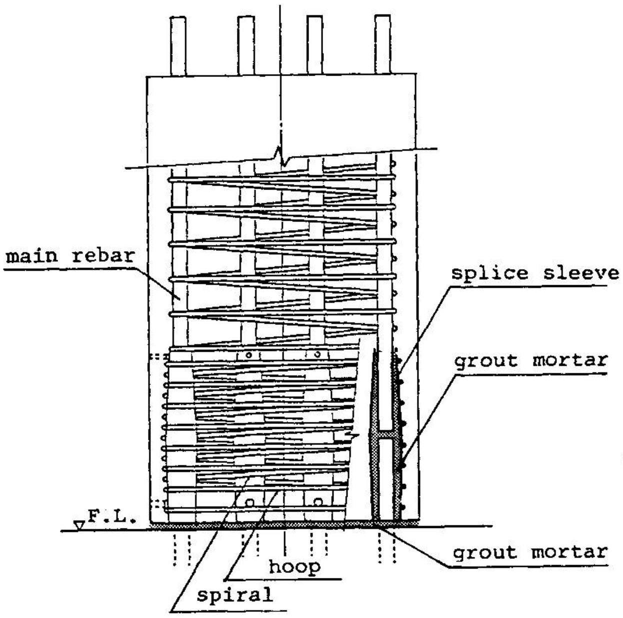


Fig.12 Pre-Cast Column

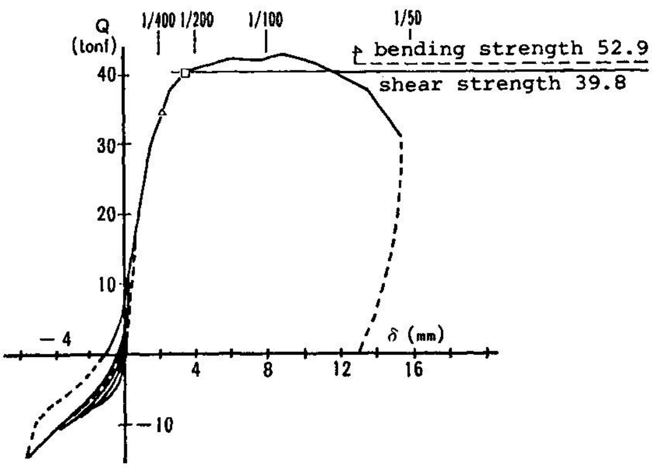


Fig.11 Load-deflection curve of No.2

5. INNOVATION IN CONSTRUCTION

Construction method in HiRC includes many sophiscated techniques such as prefabrication of reinforcing bars, U shaped anchorage, mechanical splices, integrated form work and so on. In this particular building, more over, upper storied column is to be made of pre-cast concrete as shown in Fig.12. The aim of this technique is to reduce the construction time from 8 days/floor to 7 days/floor as well as to conduct better quality control of whole works. As this application is also unprecedented in Japan, laboratory testings on structural behaviors are also conducted as well as on site testing on its practicabilities.

6. CONCLUDING REMARKS

Under very severe natural conditions, it needs long-term efforts in research and development to realization of building a tall reinforced concrete structure. Innovations in materials and methods are indispensable, but, they contribute fully to benefit not only to those concerned but also to general public. Using the methods described above, one of the urban renewal of the City of Kawasaki is to be fulfilled in coming year.

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Retrofitting of Reinforced Concrete Structures with Steel Bracing Systems

Restauration de structures en béton armé avec des contreventements métalliques

Verstärkung von Stahlbetonbauten mit Stahlverbänden

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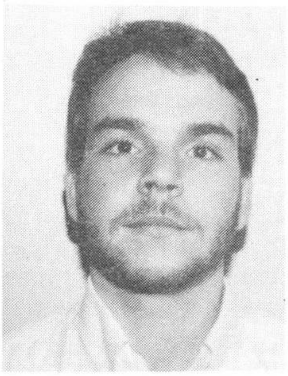

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SUMMARY

Steel bracing systems have been used for the retrofitting of reinforced concrete buildings with seismically inadequate lateral resistance. Applications of this retrofitting technique are presented and the main aspects of the design of the bracing system are discussed. The behavior under inelastic cyclic lateral deformations of a steel braced frame with weak columns is investigated with an analytical model.

RÉSUMÉ

Certaines structures en béton armé ayant présenté une mauvaise résistance latérale aux séismes ont été restaurées au moyen de contreventements métalliques. Trois applications pratiques de cette technique sont présentées et les principaux aspects du calcul du système de contreventement sont discutés. Le comportement d'un cadre contreventé soumis à des déformations latérales inélastiques est étudié à l'aide d'un modèle analytique.

ZUSAMMENFASSUNG

Für die Verstärkung von Stahlbetonbauten mit ungenügendem Horizontalwiderstand gegen Erdbebeneinwirkungen können Stahlverbände verwendet werden. Drei praktische Anwendungen dieser Technik werden vorgestellt und die prinzipiellen Aspekte der Bemessung solcher Verbände werden besprochen. Anhand eines analytischen Modells wird das Verhalten eines mit Stahlstreben verstärkten Rahmens unter nicht elastischen, zyklischen Verformungen untersucht.

1. INTRODUCTION

A large number of existing reinforced concrete frame structures are in need of seismic retrofitting because of inadequate lateral resistance. The inadequacy typically results from a poor design, a change in usage, or a change in design loads subsequent to the original construction. One promising retrofitting scheme uses diagonal steel bracing to strengthen and stiffen the structure. The bracing system is typically attached to the perimeter frames. By working on the exterior of the building, disruptions are minimized during and after construction.

2. APPLICATIONS

Steel bracing has been used for retrofitting inadequate structures as well as for repairing damaged structures. A well-publicized example is the five story Japanese school building shown in Fig. 1 [1]. The perimeter frames consist of deep spandrel beams and short columns damaged in the 1978 Tokachi-Oki earthquake. The damaged frames were retrofitted with a bracing system detailed to provide maximimum energy dissipation under cyclic inelastic loading. Retrofitting also included altering the deep spandrel beams in order to transform the brittle "weak column-strong beam" frame into a ductile "strong column-weak beam" frame.

Fig. 2 shows a twelve story medical building in Mexico City, which suffered structural damage to column and beams of the first three stories during the 1979 Petatlan earthquake and was subsequently repaired and strengthened with external steel trusses [2]. The trusses feature steel columns to resist high overturning moments. The slabs were strengthened to transmit the seismic shear forces to the new bracing system. The foundations of the perimeter frames were strengthened with steel piles. The project was completed in ten months at approximately 20 percent of the replacement cost of the building and with minimal disturbance to the users. Unlike many surrounding buildings, the retrofitted structure performed very well in the devastating 1985 earthquake.

The building of Fig. 4, the Zaragoza Hospital in Mexico City, suffered only minor damage in the 1985 earthquake. To reduce the possibility of damage in future events, diagonally braced rectangular steel frames are being added in the bays of the perimeter frames. The prefabricated steel bracing units are positioned and concrete is cast between the steel bracing unit and the existing concrete frame. Shear is transferred by dowels welded to the bracing unit and epoxy grouted into the concrete frame. The braces are square built-up sections designed to yield rather than buckle in compression.

The use of the bracing retrofit technique has also been considered in the U.S., particularly for cases similar to the Sendai School, which is typical of many buildings constructed in the 1950's in seismic zones. The behavior of a frame featuring weak short columns and strong beams retrofitted with a steel bracing system was investigated experimentally using a large scale model shown in Fig. 3 [3]. The experimental work was coupled with the analytical study presented in the next section.

3. ANALYTICAL STUDY OF A STEEL BRACED FRAME

3.1 Model

To further understanding seismic behavior of a steel braced frame, the lateral load-drift relationship under cyclic loading of a simple model was analyzed.

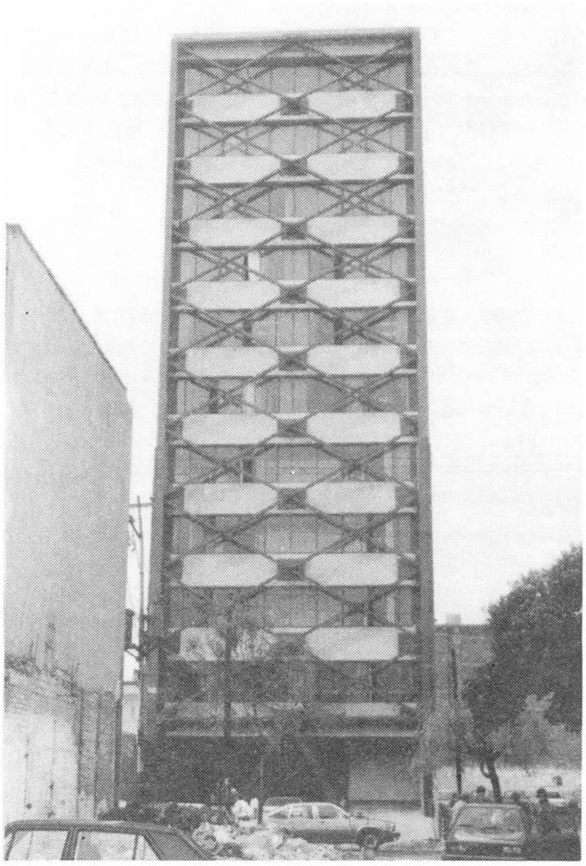
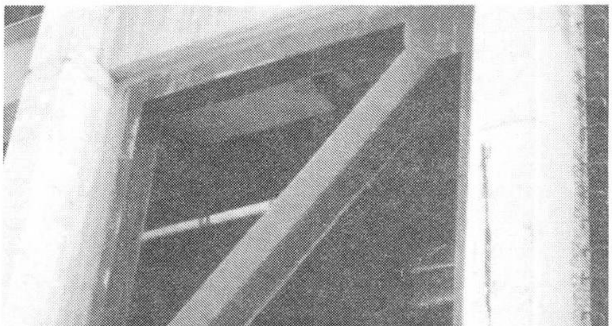


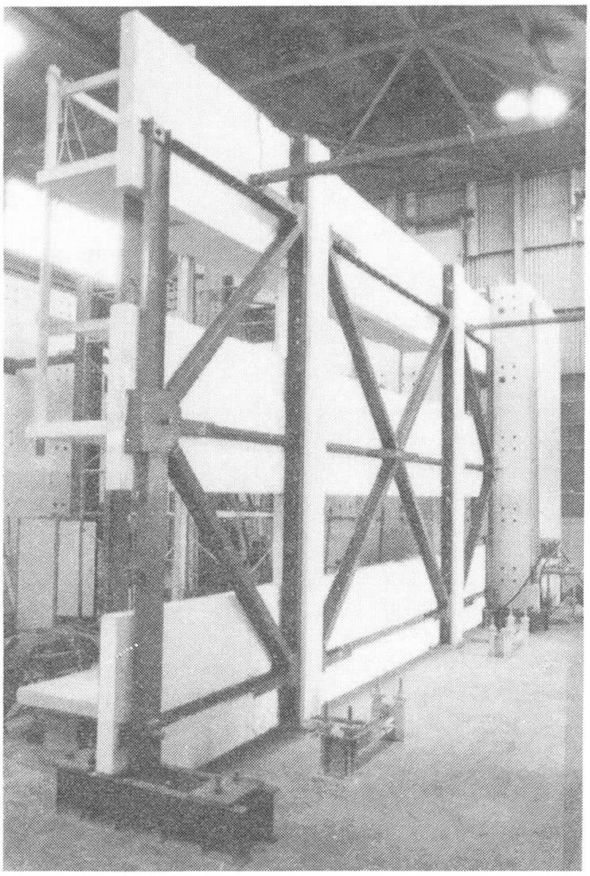
Fig. 1 Sendai School, Japan [1]

Fig. 2 Durango 49 Building, Mexico City [2,4]

Fig. 3 Experimental test of a steel braced frame [3]

Fig. 4 Zaragoza Hospital, Mex. C. [4]





The analytical model features two beams and a column simulating the frame of Fig. 5 braced as shown in the sketch in Fig. 6. The support conditions were chosen to reproduce boundary conditions of an interior column of the braced frame under lateral loading. The analytical models for the hysteretic behavior of the columns, beams, and braces were based on experimental results [5,6]. The computer program was checked using data from the experimental braced frame test.

3.2 Computed Response

In the analytical study, the slenderness ratio (kl/r) and the strength was varied and the model was analyzed for various cyclic loadings [4]. In the case described here, the braces have a slenderness of 80 and were designed to double the strength of the structure. The model was subjected to a single cycle of loading with reversal at large inelastic drift. The contribution of the frame and braces to the lateral resistance of the braced frame is shown in Fig. 5. The failure sequence is shown in Fig. 6. Important points in the response are numbered. First, the columns (1) and then the beams (3) crack. The bracing system remains elastic until the compression brace B2 buckles (4) at a drift of 0.20 percent and at 85 percent of the peak strength. The columns fail in shear (5) at a drift of 0.40 percent, the braced frame then loses strength until point 7, when the column has no lateral capacity in either loading direction. From point 7 onward, the frame no longer contributes to the lateral resistance and the bracing system controls behavior. Following the loading reversal (8), brace B1 is loaded in compression, buckles at point 9, and reaches post-buckling capacity (11) before brace B2 yields (12). When the load is reversed (13), brace B2 buckles for the second time when it reaches the post-buckling load level (14), and brace B1 yields (16) at a large drift level. The large reduction of strength and stiffness between first and

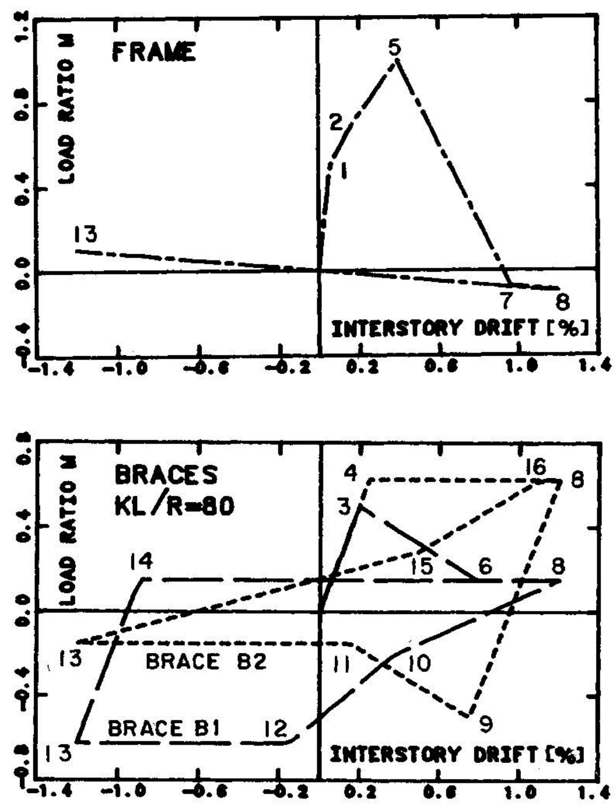


Fig. 5 Frame and bracing system under cyclic loading [4]

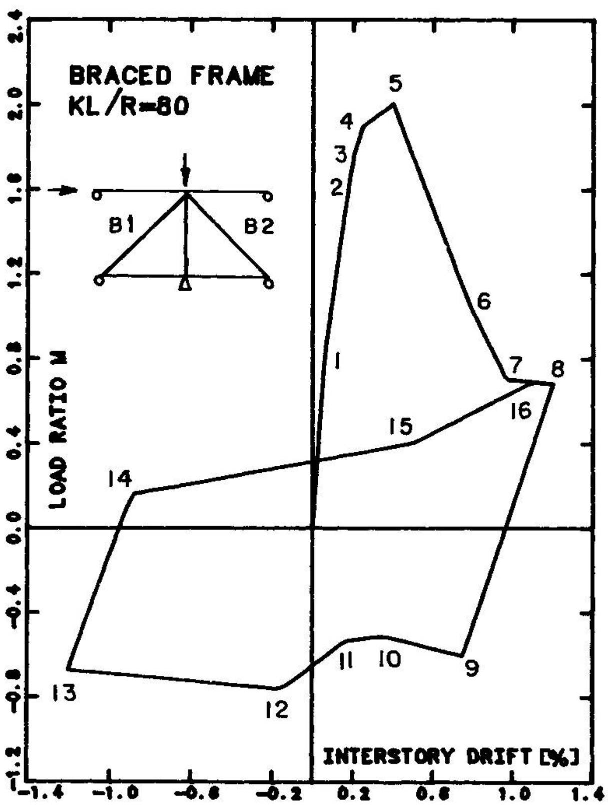


Fig. 6 Braced frame under cyclic loading [4]

second loading in the initial direction is due to loss of short column capacity and to buckling of the braces.

3.3 Behavior

Inelastic buckling of the braces is the main problem in designing a steel braced system. As was observed in experimental tests [3], the alternate buckling and yielding of the brace is linked with large local deformations at the brace connections which may lead to failure. Buckling also limits energy dissipation in the bracing system. One way to avoid inelastic buckling is to use very low slenderness ratios to guarantee that the braces yield rather than buckle in compression. Another way to prevent inelastic buckling is to use braces which buckle elastically, such as cables. The cables could be prestressed to improve the serviceability behavior. Alternatively, the bracing system may be designed to remain elastic which, in addition to preventing buckling, has the advantage of limiting drift during an earthquake. The steel bracing scheme is actually very well-suited for elastic design, since most of the added strength is in the elastic range for reasonable brace slenderness ($kl/r < 80$). The performance of the braced frame is optimized if the bracing system and the frame are well-matched in terms of their relative deformability. In Fig. 5 the two systems are well-matched, since the columns do not suffer substantial damage before they reach peak elastic strength.

4. THE RETROFITTING PROCESS

4.1 Decision to Brace a Structure

The main steps in the process leading to retrofitting a structure with a steel bracing scheme are outlined in the flowchart of Fig. 7. The evaluation of the seismic adequacy of the structure (step 1) consists of comparing performance requirements with expected behavior under seismic loads. If the structure is found inadequate (step 2), the owner must choose between retrofitting or replacement (step 3). The retrofitting scheme must be designed to correct deficiencies in the existing structure; that is, lack of strength, stiffness, or ductility (step 4). The retrofitting scheme should also be considered in terms of its impact on aesthetic qualities and on the usability of the building during and after construction. The rest of the flowchart is for the case where a steel bracing scheme best satisfies the requirements defined in step 4. Bracing may be combined with other retrofitting techniques. For example, bracing of perimeter frames may be used with column strengthening or infill shear walls in interior frames.

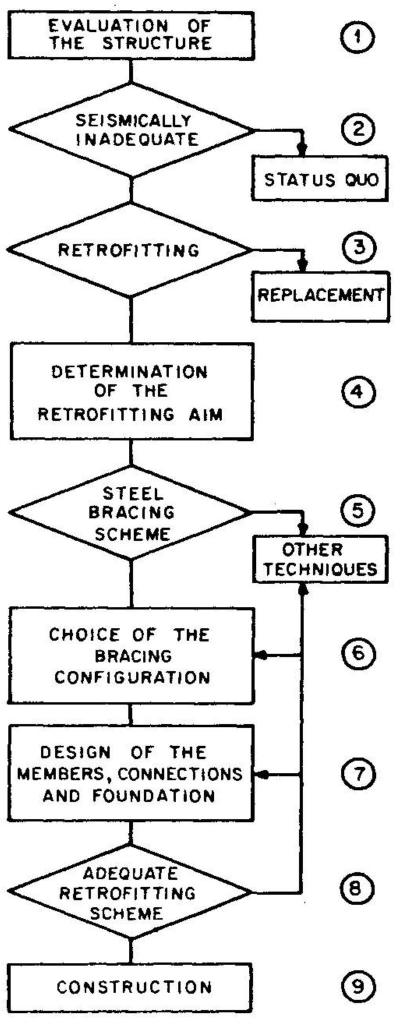


Fig. 7 Flowchart for the retrofitting process [4]

4.2 Design of the Bracing System

The choice of the bracing system configuration (step 6) includes selecting frames and bays to be braced and selecting bracing patterns.

Changes in the force distribution in the existing structure must be considered to avoid overloading certain members, or introducing torsional eccentricities in the plan of the structure, or within the braced frames. Once a configuration has been chosen, the bracing system can be designed and detailed (step 7). To maximize the drift range in which the braced frame responds elastically, brace slenderness should be low and drift levels at which the frame and the bracing system suffer significant damage should be kept as similar as possible. If columns function as vertical elements of the bracing system, they must be able to carry the additional loads. Connections of the bracing system must be detailed (welds, bolted joints) carefully to avoid local failures under inelastic cyclic deformations. The foundations of the braced frames may need strengthening because the retrofitted structure typically imposes greater forces on foundations. In the construction phase of a retrofitting scheme (step 8), allowance should be made for higher fitting tolerances and for in-situ modifications.

6. CONCLUSION

Steel bracing systems are very well-suited for retrofitting operations aimed toward strengthening and/or stiffening reinforced concrete structures with inadequate lateral resistance. The main advantage of the technique is that strength and stiffness can be adjusted to achieve a variety of design objectives because the bracing system is independent of the existing frame. The retrofitted structure can be designed to respond primarily in the elastic range, thereby limiting damage which would occur under drift in the inelastic range. Problems associated with inelastic buckling may be alleviated by using braces which buckle elastically, such as cables.

ACKNOWLEDGMENT

Support of this study by the National Science Foundation is appreciated.

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Ouvrage à âmes plissées mis en place par poussage, Charolles, France

Viadukt mit gefalteter Stegbleche und dessen Ausführung im Taktschieben verfahren

Construction by the Incremental Launching Method of a Viaduct with Corrugated Steel Plate Webs

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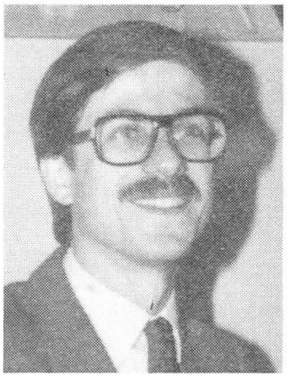

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RÉSUMÉ

Le viaduc de Maupré fait suite à la réalisation de trois ponts expérimentaux suscitée par la politique française d'innovation en génie civil. Son tablier est une poutre-caisson triangulaire dont les âmes, en tôle d'acier plissée, sont inclinées à 45 degrés pour converger vers un tube métallique rempli de béton qui constitue le hourdis inférieur. Une préflexion d'ensemble et une précontrainte centrée par câbles permettent le poussage du tablier avec ses superstructures. Des câbles extérieurs au tracé trapézoïdal complètent en service la précontrainte de l'ouvrage.

ZUSAMMENFASSUNG

Der "Viaduc de Maupré" wird als vierte Versuchsbrücke im Rahmen der französischen "Politique d'Innovation" erstellt, die Neuentwicklungen auf dem Gebiete des Bauwesens fördern soll. Den Brückenüberbau bildet ein dreieckförmiger Kastenträger, dessen Stege aus gefalteten Profilstahlblechen unter 45° geneigt sind. Diese werden auf den Untergurt, ein mit Beton gefülltes Stahlrohr, aufgeschweisst. Durch eine Gegenkrümmung des Stahlträgers vor dem Betonieren der Fahrbahnplatte und durch eine zentrische Vorspannung des Ueberbaus wird die Ausführung im Taktschiebeverfahren erleichtert Zusätzliche Spannkabel, die trapezoidal im Innern des Hohlkastens geführt sind, ergänzen die Vorspannung für die Einwirkungen im Gebrauchszustand.

SUMMARY

As part of the French innovative programme, three different solutions for bridge decks were recently investigated. One of these techniques was selected for the construction of the Maupré Viaduct. The bridge deck is a triangular box girder. The webs sloped at 45° are made of corrugated steel plate. These webs are welded on a steel tube acting as a bottom flange. This tube is grouted with concrete. Two kinds of longitudinal prestressing tendons, external to concrete, were used : straight tendons stressed span by span ; continuous tendons with a trapezoidal profile. The construction method was by incremental launching.

1. CONSTRUCTION DE PONTS EN OSSATURE MIXTE PRECONTRAINTE

Après un certain nombre d'études théoriques et expérimentales (3), on a construit en France plusieurs ponts dont les tabliers sont constitués d'une ossature mixte, comportant deux membrures en béton reliées par des âmes en acier et précontraintes par des câbles longitudinaux extérieurs.

Le passage supérieur N° 8 de l'Autoroute Orléans Clermont-Ferrand a un tablier de ce type, pour lequel les âmes sont des tôles planes de 12 mm d'épaisseur, raidies verticalement et longitudinalement. Le tablier est une travée isostatique de 38 mètres de portée. Il a été construit par Fougerolle pour la société concessionnaire de l'Autoroute : Cofiroute. L'ouvrage est en service depuis l'été 1985.

Le pont d'Arbois a un tablier dont les âmes sont remplacées par des treillis plans de type Warren à montants verticaux, constitués de profilés HEB 300 en acier. Cet ouvrage comporte trois travées de 29,85 m, 40,40 m et 29,85 m de portée. Il a été construit par Dragages et Travaux Publics et la Société Générale d'Entreprise, pour le compte de l'Etat; il est en service depuis l'été 1986.

Le pont de Cognac, lui, possède des âmes en acier constituées d'une tôle de 8 mm d'épaisseur, plissées selon un motif trapézoïdal. Cet ouvrage comporte trois travées de 31,00 m, 43,00 m et 31,00 m de portée. Il a été construit par Campenon Bernard BTP pour le compte de l'Etat. Il est en service depuis l'automne 1986.

Ces trois ouvrages, de dimensions modestes, ont permis de valider les hypothèses de calcul mises au point lors des études précédentes.

2. LE VIADUC DU VALLON DE MAUPRE, A CHAROLLES

Pour poursuivre sa politique d'incitation à l'innovation, la Direction des Routes a choisi de lancer un appel d'offres à variantes larges pour la construction du viaduc du Vallon de Maupré, à Charolles. Les caractéristiques fonctionnelles et le parti de l'ouvrage étaient imposés, et les entreprises devaient proposer uniquement des solutions "innovantes" à tablier mixte précontraint.

Ce viaduc a une longueur de 324,45 m, il comporte sept travées; la plus grande portée est de 53,55 m.

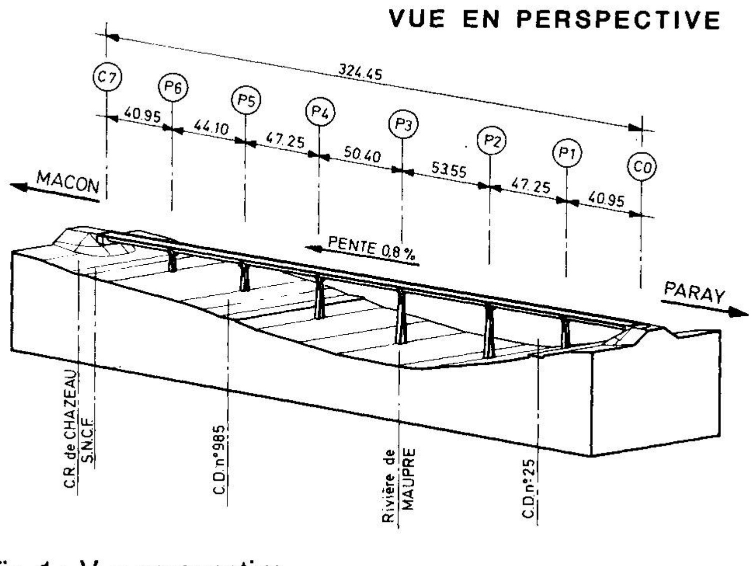

Fig. 1 : Vue perspective

Outre les deux solutions de base étudiées par l'administration, six solutions variantes ont été proposées par les entreprises. Le Maître d'Ouvrage a retenu la solution présentée par Campenon Bernard BTP : une poutre caisson triangulaire à âmes en tôle plissée.

3. LA STRUCTURE

La structure transversale du tablier du pont de Charolles est le résultat d'une évolution ayant pour origine celle du pont de Cognac. L'idée directrice a été de remplacer le hourdis inférieur en béton par une membrure en acier qui offre une résistance à la traction.

COUPE TRANSVERSALE

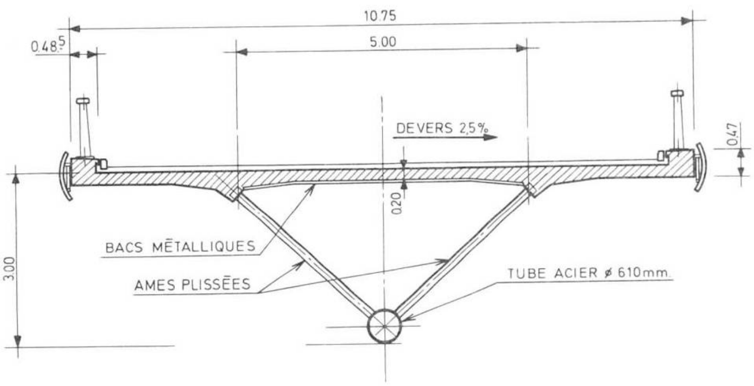

Fig. 2 : Coupe transversale

Le tube fut choisi pour son caractère de structure de révolution qui présentait une certaine souplesse du point de vue géométrique, et pour sa disponibilité directe sur le marché. Il est rempli de béton pour des raisons de déformabilité lors de l'application des réactions d'appui en cours de poussage.

La taille de ce tube, réduite parce qu'adaptée aux efforts exercés, conduit à faire converger les âmes plissées avec une inclinaison sur la verticale voisine de 45 degrés, constituant ainsi un triangle isocèle dont le hourdis supérieur en béton est la base. Cette forme a pour caractéristique de très bien fonctionner en torsion. La connexion entre âmes et hourdis supérieur est assurée par des tronçons de cornières disposées transversalement sur une platine; ces éléments sont assemblés par soudure. Le dimensionnement de cette connexion est déterminé pour la presque totalité par des dispositions de bonne construction vis à vis du comportement local.

Le V que forme la structure est soumis, sous l'action des charges verticales, à des efforts d'ouverture qui sont équilibrés par une précontrainte transversale, constituée de monotorons T15 GPE uniformément répartis dans le hourdis supérieur.

La totalité des éléments constitutifs du tablier représente, ramenée au mètre carré d'ouvrage, les quantités suivantes :

– béton : 0,27 m
– précontrainte : 19 kg
– acier passif de béton armé : 54 kg
– acier E36-4 de structure : 82 kg.

Cela correspond à un poids par mètre de structure brute courante de 7,7 t et à un poids par mètre de structure courante équipée de ses superstructures de 12 t.

4. LA MISE EN ŒUVRE

La structure métallique de l'ouvrage est préfabriquée et prépeinte en atelier par tronçons d'environ 12 mètres de longueur. Un bac métallique nervuré HI-BOND qui constituera le coffrage de la dalle centrale ferme le triangle structurel. Ces tronçons sont livrés sur chantier par camions semi-remorques et mis en position à la grue sur le banc de poussage.

Le banc est composé d'une poutre unique cintrée, à concavité vers le bas, suivant un rayon de 7 000 mètres permettant d'obtenir une préflexion du tablier lors de l'assemblage des tronçons d'ouvrage. Ces tronçons sont soudés par deux, puis par quatre pour

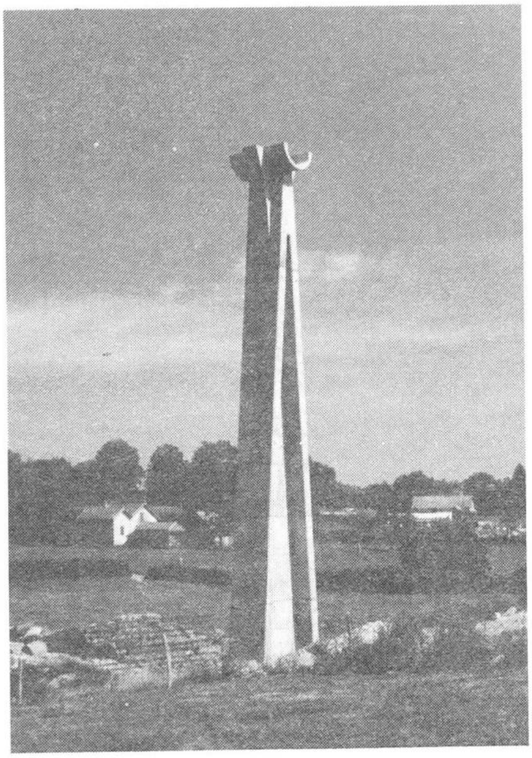

Fig. 3 : Vue d'une pile

constituer une travée. Le hourdis supérieur de cette travée est alors ferraillé et bétonné.

La précontrainte de poussage est ensuite mise en tension. On pousse la travée entière avec ses superstructures, hormis l'étanchéité et le revêtement. Il s'agit en fait d'un tirage à partir de la culée avec un effort maximum de 170 KN.

C'est la première travée de la structure qui constitue l'avant bec; pour cela le bac acier est remplacé par une tôle raidie épousant la forme du coffrage. Cet avant bec est complété par une prolongation sur 7 mètres du tube inférieur. Cet élément est équipé d'un dispositif d'accostage de grande amplitude susceptible de reprendre une flèche métrique.

En cours de poussage, la stabilisation transversale de la structure est assurée : sur le banc, par des étais roulant sur des chemins qui lui sont parallèles; sur les appuis définitifs, par des étais équipés de galets.

Les rouleurs sans cesse stabilisés transversalement supportent le tablier sur le banc. Les appuis définitifs sont équipés de profilés de 2,50 m de longueur, montés sur hydraulique, sur lesquels glissent les appareils d'appui de poussage qui épousent la forme du tube.

5. LA PRECONTRAINTE - LA PREFLEXION - LES ENTRETOISES

En plus du câblage de poussage composé de 12 à 14 câbles 6T13 centrés, régnant sur une travée d'appui à appui, l'ouvrage est précontraint longitudinalement par 4 câbles 19T15 tendus d'un bout à l'autre du tablier et ancrés dans les blocs d'about en béton.

Le câblage retenu, pseudo-funiculaire partiel des charges permanentes, permet d'obtenir pour une faible force de précontrainte des compressions suffisantes sur appui pour que le hourdis supérieur soit toujours comprimé en service.

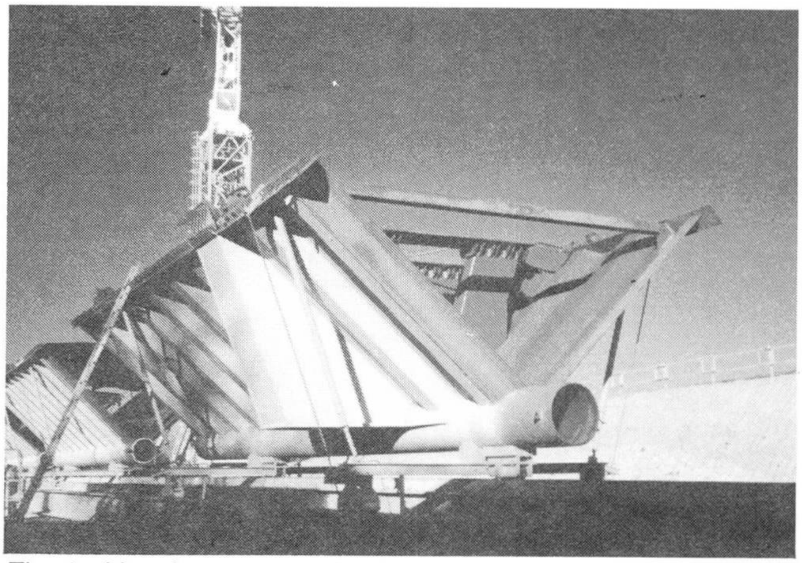

Fig. 4 : Vue d'un tronçon de charpente

Ces câbles 19T15 ont entre appuis un tracé trapézoïdal; ils changent de direction en travée, dans des déviateurs métalliques solidaires des âmes et du tube inférieur, par l'intermédiaire de tubes cintrés en acier. Les pertes de tension dues à leur grande longueur sont compensées par une opération de retension par modification du tracé. La selle de déviation sur appui central est rehaussée d'environ 55 centimètres par vérinage depuis le dessus du tablier.

L'ouvrage étant assez souple, ces câbles, considérés comme des haubans intérieurs au caisson, sont soumis à des variations de tension sous charges. Pour éliminer les risques de fatigue qui en découlent, on réalise des dispositifs qui empêchent le câble de glisser dans sa gaine au niveau des déviations.

La préflexion donnée au tablier, par la géométrie qui lui est imposée sur le banc, est une technique qui permet d'effacer les effets du retrait du hourdis supérieur en comprimant celui-ci d'environ 0,5 MPa, la traction concomitante du tube inférieur étant bénéfique sur appui, et sans conséquence en travée.

La structure triangulaire pose un problème de transmission des efforts au niveau des piles. La solution retenue consiste à adjoindre au caisson des entretoises métalliques en forme d'oreilles qui reconstituent localement l'allure d'un caisson classique à deux hourdis parallèles, transmettant ainsi les charges aux piles par l'intermédiaire d'appuis en élastomère fretté.

6. DISPOSITIF EXPERIMENTAL

L'ouvrage fait l'objet d'une expérimentation. Ce dispositif a pour objectifs le contrôle des paramètres suivants:

- répartition des contraintes de flexion longitudinale entre hourdis supérieur, âmes, tube métallique et béton du tube,
- fonctionnement du tube rempli de béton,
- déformabilité de la structure en cours de poussage et en service,
- étude des tensions dans les câbles de précontrainte de 325 mètres.

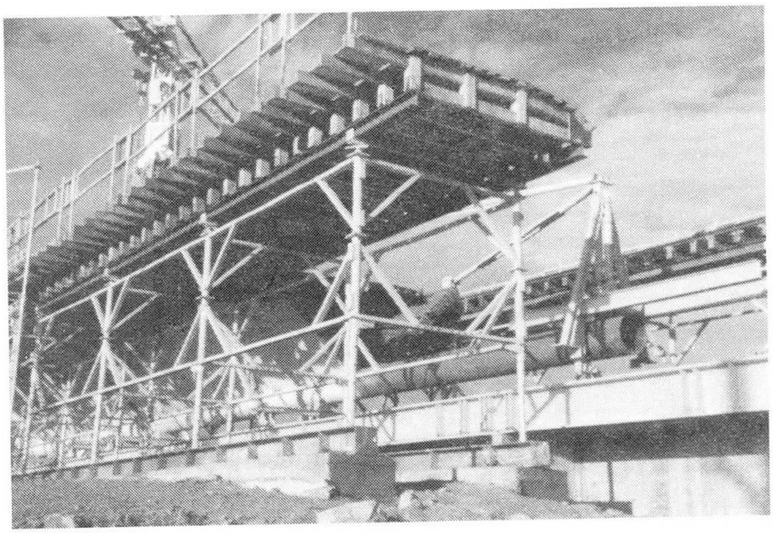

Fig. 5 : Vue du dispositif d'accostage

Dans ce but, une section a été complètement équipée de jauges de déformation collées sur les tôles ou sur les aciers passifs. L'un des appareils d'appui de poussage sera muni de pesons. Enfin des mesures de flèche de précision seront effectuées lors des essais de l'ouvrage.

7. LECONS ET PERSPECTIVES

Une opération paraissait délicate à mener à bien avant fabrication : l'accostage et le soudage de l'âme plissée sur le tube. Celle-ci s'est en fait déroulée dans d'excellentes conditions.

De manière plus générale, on peut dire que la forme triangulaire est assez coûteuse en matière, et que la méthode de construction approche de la portée limite pour ce type de structure.

Il convient néanmoins de rappeler que le poussage de la structure métallique a été réalisé avec la dalle en béton – ce qui n'est pas classique pour les ossatures mixtes traditionnelles – et de plus avec les superstructures hormis étanchéité et revêtement.

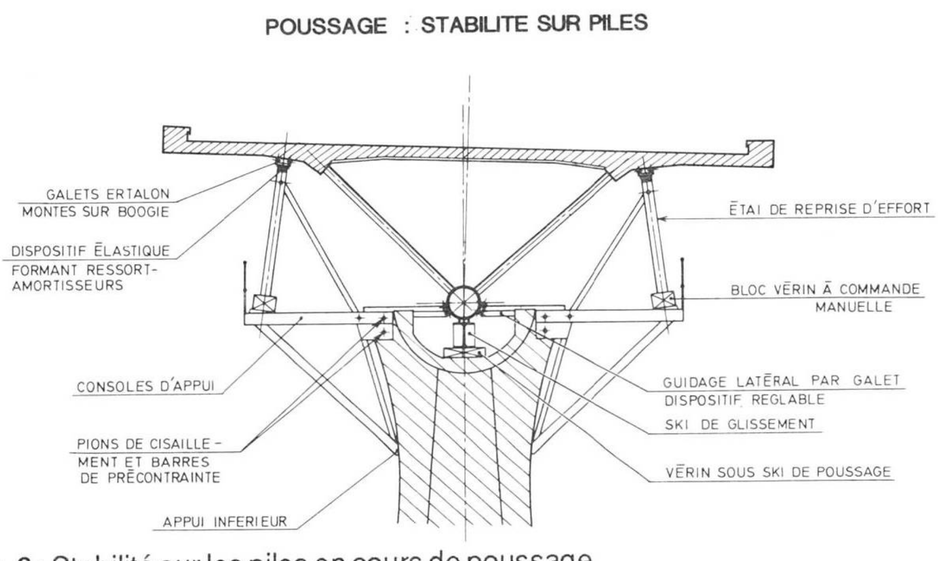

Fig. 6 : Stabilité sur les piles en cours de poussage

Les âmes plissées ont donc démontré, pour cet ouvrage poussé, comme pour le Pont de Cognac construit sur cintre, leur grande fiabilité. Il faudra mettre en œuvre de nouvelles méthodes de construction pour aborder les grandes portées, domaine naturel de ce type de structure.

Pour que cet avenir se concrétise, il est souhaitable que les maîtres d'ouvrage rendent possible la construction de ponts de grande portée de ce type, de façon à permettre aux âmes plissées de trouver leur domaine d'efficacité, et de démontrer dans ce cas leur compétitivité structurelle et économique, dans l'intérêt de tous les intervenants.

8. INTERVENANTS

- Maître d'Ouvrage : Etat, Ministère de l'Equipement, du Logement et des Transports
- Maître d'Oeuvre : Direction Départementale de l'Equipement de Saône et Loire
- Auteur de la conception et du projet d'exécution : Bureau d'Etude de Campenon Bernard BTP
- Contrôle du projet et assistance technique du Maître d'Oeuvre : SETRA et CETE de Lyon.
- Entreprise : Campenon Bernard BTP - Direction des Ouvrages d'Art Spéciaux
- Sous-traitant chargé de l'exécution des parties métalliques : Strasbourg Entreprises
- Sous-traitant chargé de l'exécution des piles et culées : Maillard et Duclos
- Sous-traitant chargé de l'exécution des pieux : SEPICOS
- Dispositif expérimental : LCPC et LRPC d'Autun

COUPE LONGITUDINALE-TYPE D'UNE DEMI-TRAVEE

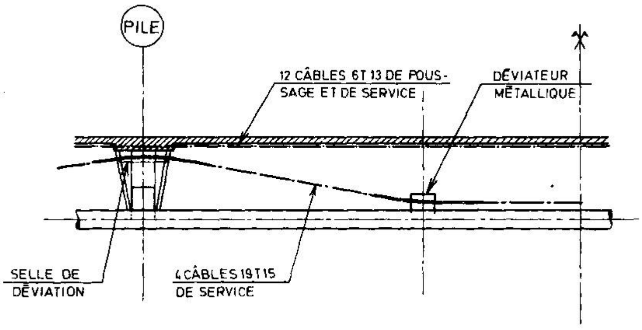

Fig. 7 : Coupe longitudinale type d'une demi-travée

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Composite Structure of Concrete and Steel Plate

Structure composite en béton et tôle d'acier

Verbundbauteile aus Beton und Stahlplatten

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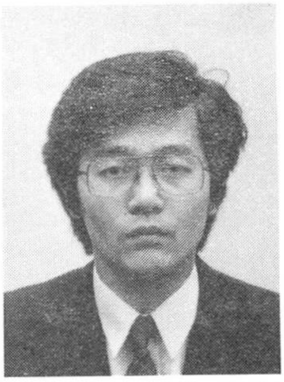

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SUMMARY
The composite structure of concrete and steel plate without reinforcing bar has various advantages compared with reinforced concrete. The steel plate, which acts also as the formwork, resists external forces together with the concrete. The rib used as the stiffener to ensure the rigidity of formwork acts as the shear connector. Use of a flanged rib improves the integrity of the concrete and steel plate. Experiments with composite beams using ribs as shear connectors revealed some fundamental factors relating to rib shape and rib arrangement in design of composite structure member, the results of which are presented.

RÉSUMÉ
La structure composite en béton et tôle d'acier, sans barres d'armature, présente quelques avantages par rapport au béton armé. La tôle d'acier, qui agit aussi comme coffrage, résiste aux forces extérieures en collaboration avec le béton. La nervure, utilisée comme raidisseur pour assurer la rigidité du coffrage, agit aussi comme goujon. L'emploi d'une aile nervurée améliore le comportement de l'élément mixte. Des expériences réalisées avec des poutres composites utilisant les nervures comme goujons ont montré l'importance fondamentale de l'emplacement et de la forme de ces nervures. Les résultats en sont présentés.

ZUSAMMENFASSUNG
Verbundbauteile bestehend aus einer Stahlplatte und unbewehrtem Beton haben verschiedene Vorteile gegenüber normal bewehrten Stahlbetonbauteilen. Die Stahlplatte dient auch als Schalung. Die aufgeschweissten Steifer geben der Stahlplatte eine grössere Steifigkeit für ihre Funktion als Schalung und dienen gleichzeitig als Schubdübel im Verbund mit dem Beton. Die Verwendung von Rippen mit Flansch verbessert das Verbundverhalten. Versuche mit solchen Trägern zeigten die fundamentale Bedeutung von Rippenanordnung und Rippenform auf. Die Ergebnisse dieser Versuche werden besprochen.

1. INTRODUCTION

In Japan, with the increase in urban population and to make better use of space, need of expanding cities toward coast or even to offshore, enlarging the unit of construction, and building larger structures by demolishing old ones. On the other hand, in the case of construction work in and around the city, not only usual safety and non-interference of urban lives are required but, especially in Japan, lack of on-site preparation area has become a matter of grave concern.

From these points of view, as well as the development of new efficient construction methods, the development and plans of new type structures have increased. For instance, lately, the submerged sewage treatment plant of reinforced concrete structure was constructed by improving the former technique of the submerged tunnel. The erection of steel parts and the concrete casting of bottom slab were carried out at a shipyard, and after flooding, the concrete of wall and top slab was cast with the body floating. The caisson thus completed was towed by tugboats from the shipyard to the project site, and was placed on the previously built seabed foundation.

The composite structure of concrete and steel plate concerned in this paper has various advantages and problems that are not seen in the reinforced concrete structure or the steel structure, as shown in Table 1. These advantages should make this kind of composite particularly suited for the following structures : (1) offshore structures, submerged structures, or bridge slabs, (2) tanks, vessels, or containers that needed tightness, and (3) structures carrying huge loads.

The greatest advantage of the composite structure over reinforced concrete is that the steel plate itself acts as the formwork, whose rigidity is further reinforced by the ribs, which act not only as shear connectors but also as stiffeners. All these combined make the structure economical.

The authors have carried out experiments on the composite beam using rib as the shear connector to create a composite action between concrete and steel plate. Also, some fundamental considerations related to rib shape and rib arrangement in design of composite structure member are presented.

2. DESIGN PROCEDURE

The design procedure of composite structure is shown in Fig. 1. Firstly, the steel shell (steel plate thickness, dimension and arrangement of stiffeners and web plates, etc.), is designed against the construction load, which consists in the external load, including the load from unhardened concrete, and the dead load. Of course, the construction load depends on the construction method. Nextly, the composite functions are reviewed with regard to the efficacy of compositioning, the efficiencies of stiffeners and the total strength against the service loads the composite is expected to carry, and the durability.

Should any of these be judged unsatisfactory, the design work is re-started from the steel shell designing again.

Table 1 Advantages and problems of composite structure compared with reinforced concrete structure

	Advantages		Problems
Construction	1. Formworks are not necessary. 2. Steel shell can be manufactured in a factory. 3. Bar arrangement is not necessary. 4. Concrete can be cast at the sea.	*Economy *Safety *High accuracy *Short of construction term *Reduction of land for construction	1. Stiffeners(ribs) are necessary to ensure the the rigidity of steel shell, making it rather difficult to fill up the space under the stiffener with concrete. 2. Calculation of stresses operating during construction is complicated. *Residual stress due to construction work must be considered as the permanent stress. *The order of concrete casting must be planned carefully.
Service-ability	1. The outer steel plate ensures the tightness 2. Concrete cover is not necessary. 3. Concrete does not fall off.	*Economy *Safety *Lightweight	1. There is possibility of increased deflection. 2. Buckling of the compression plate. 3. Corrosion of steel plate (durability).
Strength and Structural Behavior	1. High toughness with the sandwich type structure. 2. Concrete is confined by the steel plate.		1. The design of proper shape and arrangement of the stiffener (ribs) as shear connector is difficult. 2. The design method of the imperfect composite structure has not been established. 3. Influences of creep and shrinkage of concrete are not well known.

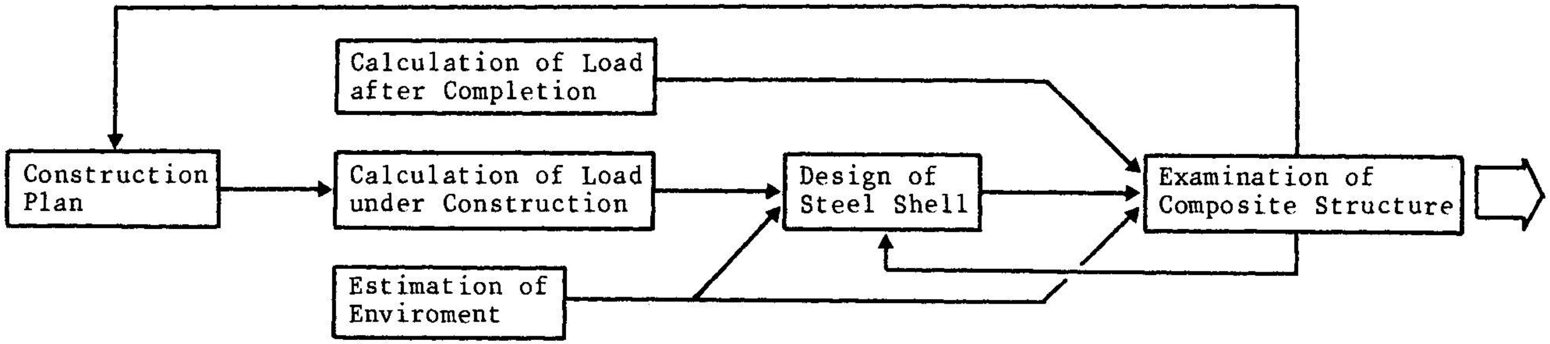


Fig. 1 Design Procedure

3. CONDITION FOR COMPOSITE FUNCTION AND STRUCTURAL BEHAVIOR

3.1 Shape and arrangement of rib

Since the shear is transferred between concrete and steel in the composite structure by the shear connector, the shape and the arrangement of shear connectors affect significantly the composite action. Since the stresses around the shear connector generally are highly concentrated, local copmressional failure or cracking of concrete may be likely. Therefore, it is realized that the shear strength may deteriorate markedly if the shape or arrangement of the shear connector is not correct.

3.1.1 Consideration on rib height for the composite action

Rib height should be greater than the largest aggregate in the concrete, and the compressive stress on the concrete enclosed in a rib should be smaller than the bearing strength.

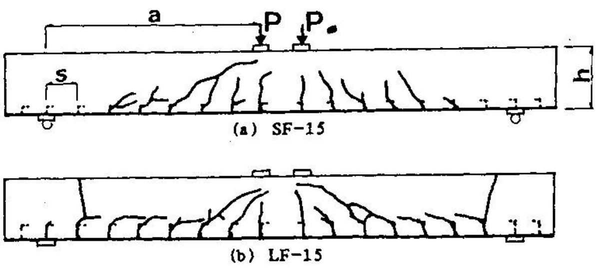
In the case that ribs are anchored in the tension zone of concrete, the rib height has no effect on the strength of the member as a whole and on the composite function (Fig. 2). But, when the rib height is too small, the concrete enclosed in a rib will fail under the bearing stress, and ribs will not work as the shear connector.

To demonstrate an extreme case, an experiment was carried out on a composite beam made up of concrete and checkered plate. The result obtained was that the beam failed simultaneously with occurrence of bending crack, and the concrete in contact with the checkered plate was pulverized at the places of stress concentration.

3.1.2 Necessity of rib-flange

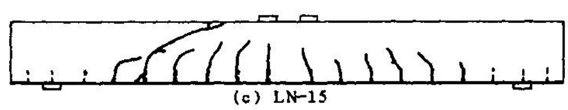
The rib-flange is necessary for the rib to act as the shear connector. In general, the flange width should be larger than the size of the greatest aggregate.

The shear connector should prevent the separation of concrete and steel plate, besides transferring the shearing force between them. If there is no flange on the rib, the bending crack from the top of rib is liable to occur early, and this bending crack will induce the diagonal tension crack in concrete (Fig. 2). From the experiments, the static shear strengths of beams with flanged ribs were higher than those of beams with unflanged ribs. On the other hand, however, the rib-flange is liable to induce almost horizontal cracks at the top of the flange, degrading the composite characteristic. And, when these horizontal cracks are linked together, the member will collapse (Fig. 2). Since the crack pattern is different with or without the rib-flange, the designer must judge which crack pattern is more harmful to the structure.



(a) SF-15

(b) LF-15



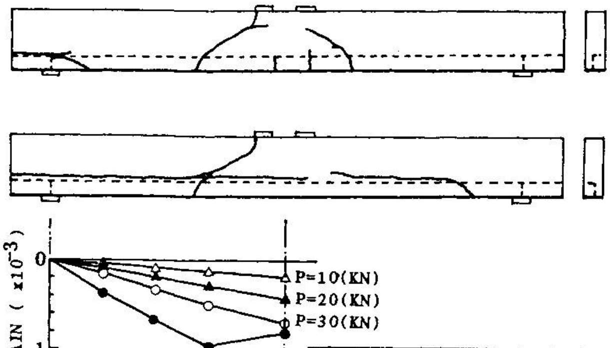
(c) LN-15

Specimen	Pmax (KN)	Rib (cm) Height	Flange	Space	a (cm)	h (cm)	f'c (MP)
SF-15	26.5	4.0	4.0	15.0	105	30	21.6
LF-15	29.4	7.5	4.0	15.0	105	30	22.1
LN-15	26.4	7.5	0	15.0	105	30	22.2

Fig.2 Effects of Rib height and Rib-flange on the strength and composite action

3.1.3 Effects of longitudinal rib

If the longitudinal rib has a proper flange, it functions as an excellent shear connector. However, as shown in Fig. 3, on account of tensile stress acting between concrete and rib-flange, it will give rise to a continuous weak plane along the rib-flange. When a crack propagates along this plane, the rib will lose its power.



Pmax (KN)	Rib (cm) Height	Flange	f'c (MP)
35.6	7.5	4.0	21.8

Fig. 3 Connection effects of longitudinal rib

3.1.4 Cares to be exercised on using longitudinal rib with transverse rib

If the longitudinal rib and the transverse rib are used together, concrete will be subjected to tensile forces from both the longitudinal and the transverse rib flanges. Therefore, their heights should not be the same, and the difference between the two need to be larger than the size of the greatest aggregate.

3.1.5 Shear reinforcement effects of transverse rib

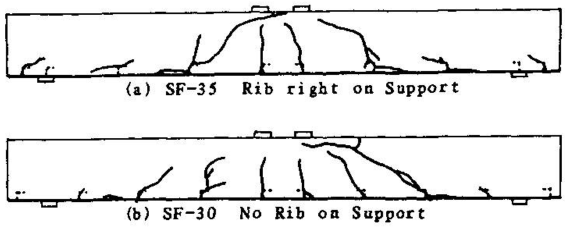
If the transverse ribs with flange are anchored in compressive zone of concrete, or the compression plate and the tension plate are connected by the transverse ribs (web), these ribs act as shear reinforcements as well as shear connectors. The shear mechanism in the member is like that in the truss action in which diagonal compressive struts equilibrate with transverse tensile ribs. Therefore, shear strength of this type of member is determined by the compressive strength of concrete or the tensile strength of transverse rib.

3.1.6 Shear reinforcement effects of longitudinal rib

If the longitudinal ribs with flange are anchored in compressive zone, or the compression plate and the tension plate are connected by the longitudinal ribs (web), these ribs also act as shear reinforcements. Vertical tensile stress in the rib varies like that of shear reinforcement (stirrup) of reinforced concrete with load increasing. But, in the case of longitudinal rib, the rib should carry the shear force (stress) from tension plate. Therefore, at the time of calculating the shear strength of longitudinal rib (Vs), the width of rib should be considered in assessing the influence of shear stress.

3.1.7 Effects of location and arrangement of transverse rib

In the case of large shear span members, the shear strength is equal to the diagonal tension crack load. And, in the composite member, cracking occurs from the top of all ribs. Therefore, when ribs are located so that the crack pattern is similar to that of reinforced concrete, the shear strength is equal to that of the reinforced concrete. As shown in Fig. 2, however, in the case the rib spacing is too short, the almost horizontal cracks are linked together easily, and the



Specimen	Pmax (KN)	Rib (cm) Height	Flange	Space	f'c (MP)
SF-35	35.2	7.5	4.0	35.0	25.4
SF-30	27.8	7.5	4.0	30.0	20.8

Fig. 4 Effects of rib arrangement on shear strength

composite action is lost early. On the other hand, in the case there exist a rib right on a support and few ribs are inside the shear span, compressive struts are formed in concrete from underneath of the load points to the supports, and the shear strength becomes larger than that of reinforced concrete, because the diagonal crack does not occur easily (Fig. 4). The mechanism of force transfer in this case is like in the tied-arch action with the concrete acting as the arch rib and the steel plate as the tie. But, as shown in 3.2, the deflection of member will be larger than that of reinforced concrete.

3.2 Deformation behavior

Deflection of composite member has been calculated by modifying the beam theory by rib spacing.

The bending stiffness of composite member under concentrated load will be $1/(1+s/a)$ as large as that calculated from the beam theory on the reinforced concrete (Fig. 5). Where, s is the rib space, and a the shear span.

From the experiments, the bond stress between concrete and steel plate becomes zero just about the time the flexure crack occur, and the axial strain of steel plate between two ribs becomes the same. Based on experimental results, the axial strain of steel plate can be assumed to be the solid line in Fig. 6 . The average (total) strain of steel plate in shear span is $(1+s/a)$ times the broken line value (perfect composite). Therefore, the deflection of this composite type member shall be $(1+s/a)$ as large as the perfect composite.

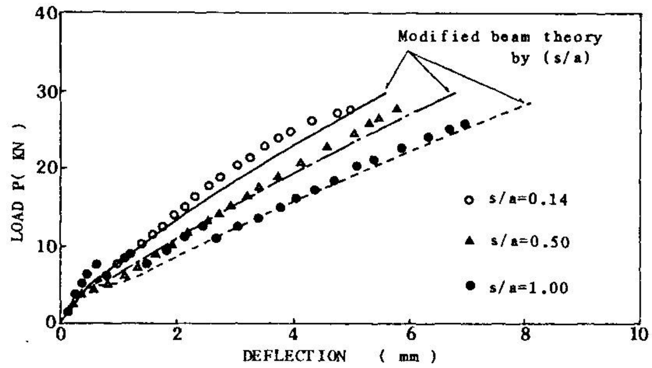


Fig. 5 Load - Deflection curve of composite beam

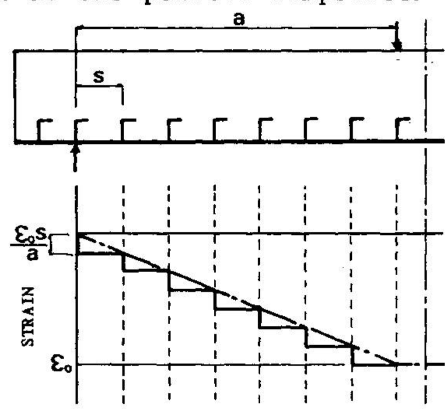


Fig. 6 Axial strain distribution of steel plate

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Concrete Filled Steel Bearing Walls

Murs porteurs constitués de parois d'acier et de béton

Tragwände in Stahlzellenverbundbauweise

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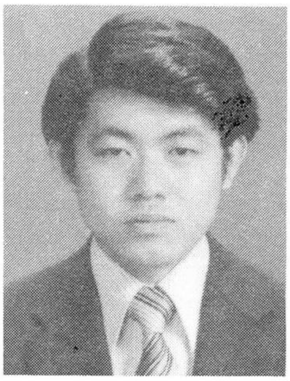

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SUMMARY
SC bearings walls are composed of box steel(S) and filled concrete (C). They are expected to reduce construction time and labor compared with reinforced concrete bearing walls with especially complicated re-bars. However, it is necessary to use comparatively thin steel plates for SC bearing walls from an economical view point. The objective of this investigation is to clarify the structural behavior of SC bearing walls with thin steel plates by axial and shear loading tests.

RÉSUMÉ
Les murs porteurs SC se composent du coffrage d'acier (S) rempli de béton (C). Comparés aux murs porteurs de béton armé avec des barres de répartition particulièrement compliquées, ils permettent de réduire le temps de construction et la main d'œuvre nécessaire. Mais pour des raisons économiques, il serait souhaitable de renforcer ces coffrages par de minces tôles d'acier. Le but de cette recherche est de déterminer le comportement structural de ces murs porteurs SC comportant de minces plaques d'acier par des tests de charges axiales et de cisaillement.

ZUSAMMENFASSUNG
Tragwände in Stahlzellenverbundbauweise bestehen aus einer mit Beton verfüllten Stahlzelle. Man erwartet von dieser Bauweise verkürzte Bauzeiten und eine Einsparung an Arbeitskräften im Vergleich zu den herkömmlichen Tragwänden aus Stahlbeton mit besonders komplizierter Bewehrung. Für eine Kostenersparnis ist allerdings die Verwendung verhältnismässig dünner Stahlplatten notwendig. Das Ziel dieser Untersuchung ist die Klärung des Tragverhaltens solcher Tragwände unter Druck- und Schubbeanspruchung.

1. INTRODUCTION

The concrete-filled steel (SC) bearing walls consist of elements of steel plate box units filled with concrete. Their structural behavior can be characterized by high strength and sufficient ductility owing to the composite effect of the steel plate and the concrete. By applying SC instead of reinforced concrete (RC) for structural constructions it can be foreseen that the arduous task of arranging and placing of rebars are eliminated, which should result in reducing construction time. This SC structural method should prove valuable to countries such as Japan which is located in high seismicity zone and is subjected to large seismic loads.

Regarding the SC bearing wall, the researches by Kato, Suzuki, et al.[1)2)] has confirmed, by basing on tests, that the relationship between loads and displacements show sufficient ductility when steel plates with width-thickness ratio of about 100 is used. They also show that the maximum shear strength could be estimated by an ultimate limit analysis.

The SC bearing walls proposed by the authors are constructed by assembling welded box steel units and filling the inside with concrete (see Fig. 1). The box units are composed of comparatively thin steel plates (width-thickness ratio of 200) in view of economical considerations. However, anxieties did exist that the composite effect to the SC bearing wall may not be adequate due to buckling of the thin steel plates at an early stress stage. In this paper, the authors describe the elasto-plastic structural behavior of the proposed SC bearing wall units by applying stiffening to the thin steel plates such as insert plates, L-shaped steel angle members and headed studs in loading tests. The tests conducted were compressive loadings and shear loadings.

2. OUTLINE OF THE PROPOSED SC BEARING WALLS

Process of the proposed SC bearing walls are 1) prefabricating a box unit consisting of two-surface plates and vertical and horinzontal insert plates, 2) assembling box units to the required length and width of the wall, 3) welding the joints of the boxes, and 4) filling the inside of the box units with concrete. The thickness of the proposed SC bearing wall is 1.25 m and the thickness of the steel plates are 12 mm. This possess equivalent strength and economy as a RC bearing wall which has wall thickness of 2 m and rebar reinforcement ratio of 1.2%. The outer dimensions of the box steel unit, which was determined in view of the convenience of construction and transportation, has the width of 2.5 m, and the height of 3 m (see Figs. 1 and 2). As a bearing wall, the proposed SC bearing wall has a smaller self-load than a RC bearing wall used in the same place because of its smaller cross section. Thus can contribute to reduce seismic loads.

The methods devised to prevent early stage buckling of the surface plate are as follows: 1) place one or two vertical insert plates in appropriate places

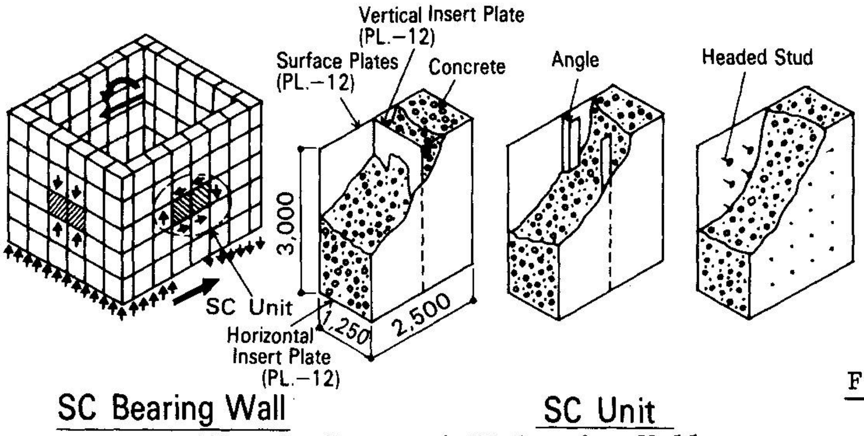


Fig. 1 Proposed SC Bearing Wall

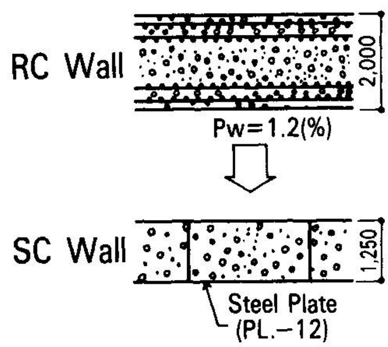


Fig. 2 Comparison of RC Wall and SC Wall

within the unit; 2) stiffen the surface plates by attaching four L-shaped steel angle members instead of the two vertical insert plates; 3) weld headed studs on the surface plates to conform with the AIJ standard for structural design of steel structures (width-thickness ratio ≤ 41)[3]; etc.

3. STRUCTURAL BEHAVIOR OF THIN STEEL PLATE SC BEARING WALLS

3.1 Outline of Loading Test Program

Table 1 and Figs. 3 and 4 show the specimens and their shapes. The specimens are approximately 1/4 scale-models of the proposed units of the SC wall. Seven respective specimens were tested each for compressive loadings and shear loadings to study the test parameters as follows.

- the composite effect of the steel plate and the concrete
- the effect of the insert plates interval (width-thickness ratio of the surface plates)
- the effect of the stiffening method to the surface plates

Table 2 and 3 show the mechanical properties of the materials.

3.2 Test Results

Figs. 5 and 6 show the compression and shear test results for each parameter. Table 4 show the final state of the surface plates and the filled concrete. Table 5 show the maximum strengths of the specimens.

Specimen	Cross Section	Structure	Width Thickness Ratio	Stiffening Method
Non Stiffening 200K		SC	200	Non Stiffening
One Insert Plate 100k		SC	100	Insert Plate (1-PL.-3.2)
Two Insert Plates 67K		SC	67	Inser Plate (2-PL.-3.2)
Angle 67A		SC	67	Angle (4-L50X 50X4)
Stud 35S		SC	35	Headed Stud (ϕ6, ℓ=40, @80)
Steel 100S		Steel	100	Insert Plate (1-PL.-3.2)
Concrete 0C(Com.) 1C(Shear)	0C 1C	Concrete	—	—

※Concrete : Fc240 Surface Plate : PL.-3.2
Side Plate : PL.-22(Com.), PL-19(Shear)

Table 1 List of Specimens

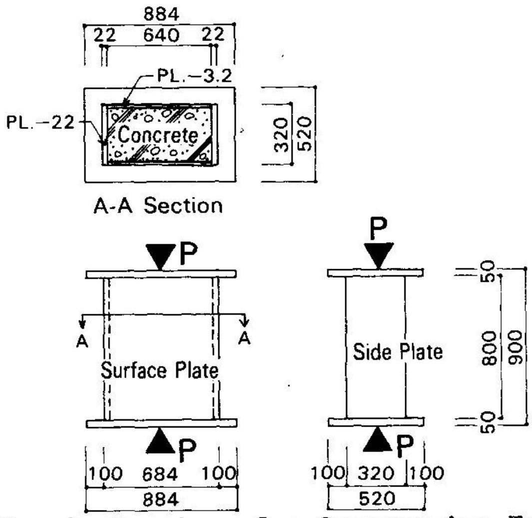


Fig. 3 Specimen for Compression Test

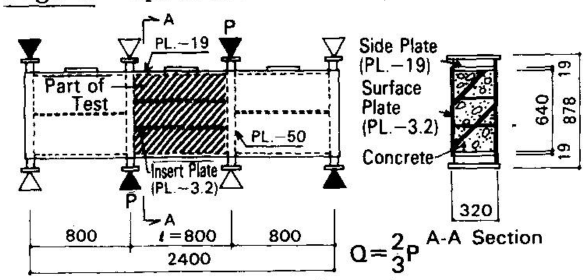


Fig. 4 Specimen for Shear Test

Thickness (mm)	Yield Stress σ_y(MPa)	Tensile Strength σ_u(MPa)	Elongation ε(%)
3.2	253	402	30.5
19	352	514	27.9
22	343	515	27.5

Table 2 Mechanical Properties of Steel

Test	Compressive Strength (MPa)	Tensile Strength (MPa)	Elastic Modulus (kN/mm²)	Age (days)
Compression	23.3	2.7	21.5	47
Shear	23.4	2.4	23.0	72

Table 3 Mechanical Properties of Concrete

3.2.1 Composite Effects of Steel Plate and Concrete (see Figs. 5(a) and 6(a))

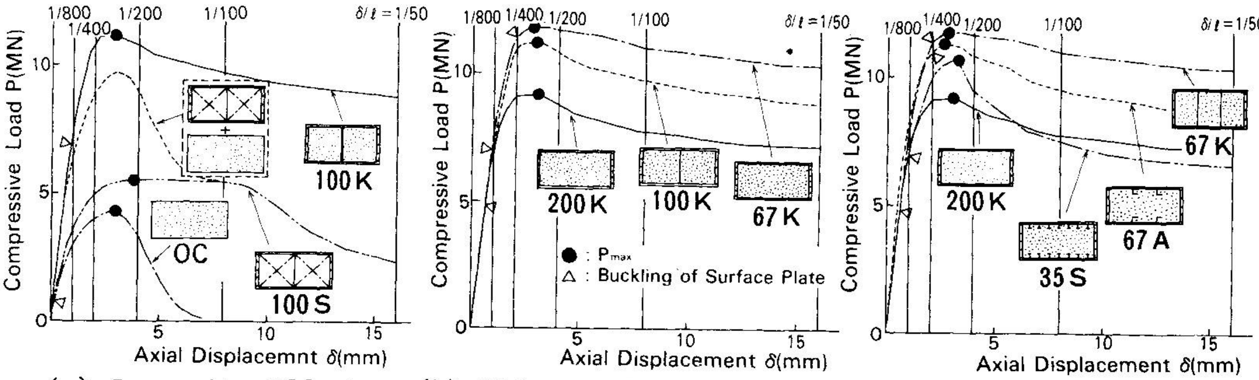
When compared with the superposed strength of the steel (100S) and the concrete (OC, 1C) specimens, the maximum strength of the SC specimen (100K) was stronger by 1.14 times in the compression test, and by 1.40 times in the shear test. The ductility was excellent in both tests.

3.2.2 Effects of Insert Plate (see Figs. 5(b) and 6(b), Table 4)

When three specimens with varying insert plate intervals (200K, 100K, 67K) were compared, their elastic rigidities were almost the same. In the compression tests, the maximum strengths were different due to the number of insert plates which support divided parts of compressive force, and due to the stiffening effects to the surface plates which increase the strengths against bucklings. In the shear tests, the maximum strengths were almost the same, but as the insert plate interval decreased, the rigidity beyond the elastic range decreased, and the displacement at the maximum strength increased. It was thought that each of the concrete blocks divided by the insert plate were deformed separately. In both tests, the early stage buckling of the surface plates had little affect on the relationship between the loads and the displacements.

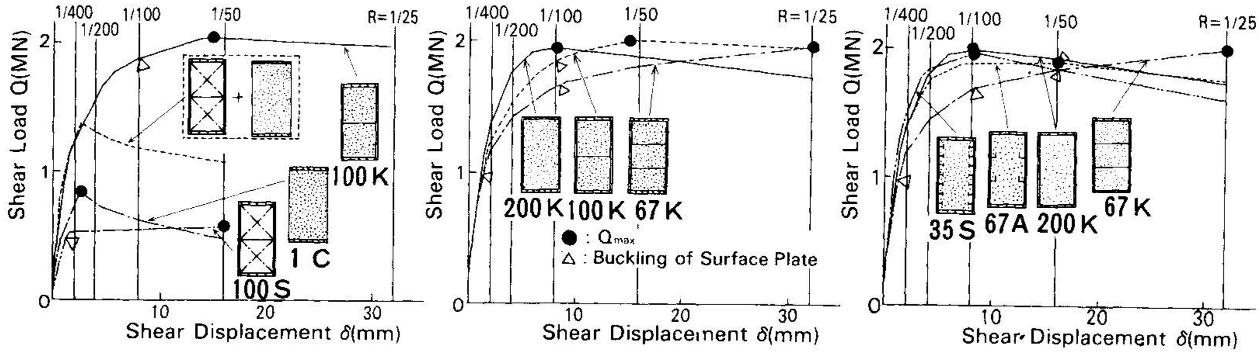
3.2.3 Effect of Stiffening Method (see Figs. 5(c) and 6(c))

In compression tests, the maximum strengths were different from aforementioned. In the relationship between loads and displacements of the shear tests, the non stiffening type (200K), the L-steel angle type (67A) and the stud type (35S) were about the same, but the two insert plates type (67K) showed a different behavior. In the case of stiffening by headed studs, the load carrying ability beyond the displacement at maximum strength was reduced earlier than others due to the failure of the concrete in between the stud head and the surface plate.



(a) Composite Effects (b) Effects of Insert Plate (c) Stiffening Methods

Fig. 5 Load-Displacement Relationship for Each Parameter in Compression Tests



(a) Composite Effects (b) Effects of Insert Plate (c) Stiffening Methods

Fig. 6 Load-Displacement Relationship for Each Parameter in Shear Tests

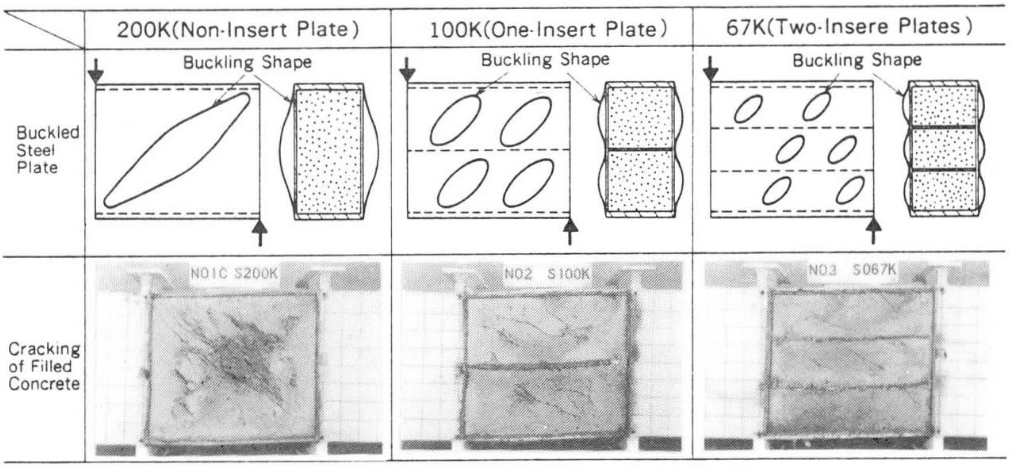


Table 4 Final State of the Steel Plate and the Filled Concrete

Specimen	Compression Test	Compression Calculated		Shear Test	Shear Calculated			
	Pe(MN)	Pc(MN)	Pe/Pc	Qe(MN)	Q_1(MN)	Qe/Q_1	Q_2(MN)	Qe/Q_2
200K	9.15	9.56	0.96	1.97	1.74	1.13	1.07	1.84
100K	11.30	10.20	1.11	2.04	1.74	1.17	1.07	1.91
67K	11.80	10.90	1.08	2.00	1.74	1.15	1.07	1.87
67A	11.40	10.80	1.06	1.89	1.74	1.09	1.07	1.77
35S	10.70	10.50	1.02	1.95	1.74	1.12	1.07	1.82
100S	5.44	5.00	1.09	0.58				
0C,1C	4.31	4.78	0.90	0.85				

$Pc = As\cdot\sigma_{cr} + (Ap+Ar)\sigma_y + Ac\cdot Fc$, $\sigma_{cr} = k\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2$, k=6.98(SC Specimen), k=4.00(100S)

Q_1 : Kato and Suzuki's Analysis $Q_2 = As\cdot\tau_s + Ac\cdot\tau_c$, $\tau_s = \sigma_y/\sqrt{3}$, $\tau_c = 0.1Fc$

As,Ap,Ar,Ac : Area of Surface Plate, Side Plate, Stiffening Member, Concrete

Table 5 Maximum Strength

3.2.4 Maximum Strength (see Table 5)

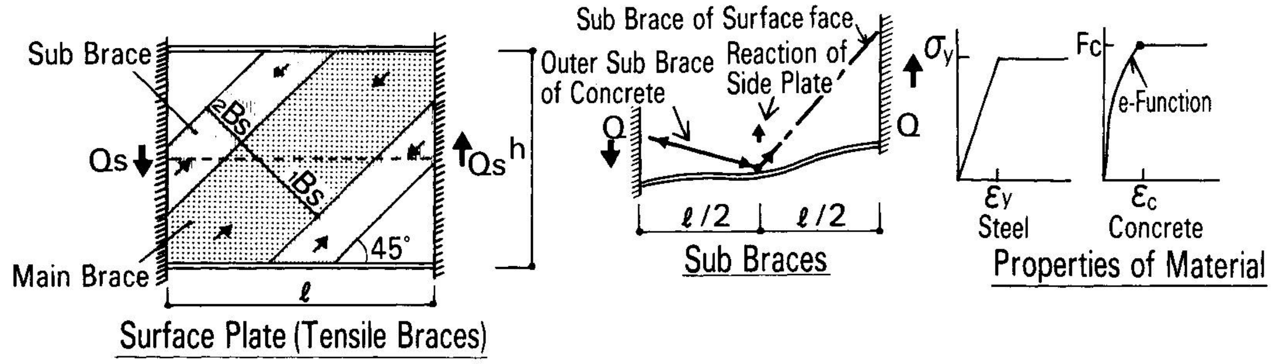
The calculated maximum strength (Pc, Q_2) of the SC bearing walls, obtained from superposed values of calculated steel strength and calculated concrete strength showed similar values with the rsults of the compression tests. The ratio of (Pe/Pc) were in the range of 0.96 ~ 1.11. But in shear tests, the ratio of (Qe/Q_2) showed safety margins of 1.77 ~ 1.91. The maximum shear strength Q_1 by Kato and Suzuki's theory showed that the (Qe/Q_1) ratios were 1.09 ~ 1.17 indicating good agreement with the shear test values.

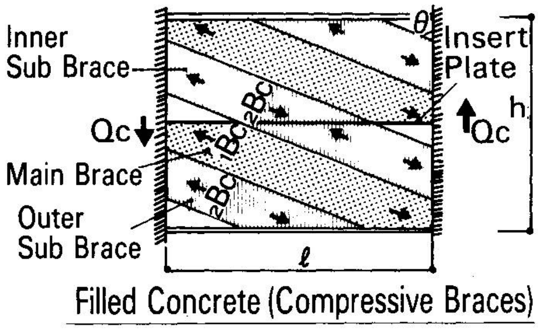
3.3 Analysis of Load-Displacement Relationship under Shear Load

Shear test results for differing insert plate intervals showed differences in the rigidity beyond the elastic range and displacement at the maximum strength. Therefore the analysis of relationship between load and displacement described below was carried out.

The analytical model was replaced by 45° direction tensile braces for surface plates, and diagonal direction compressive braces for separated concrete (in rectangle block formed by side plate and insert plates, see Fig. 7). The angle of each brace was determined by observation of the steel buckling pattern and the concrete cracking pattern after the tests (see Table 4).

The results of analysis agreed approximately with the test results (see Fig. 8). From the analyzed results, it can be seen that the maximum strength is dependent on the compressive strength of the main concrete braces, and the more the angle of the concrete braces increase due to the insert plate intervals, the more shear displacement at compressive strength increased. And also, the analytical elastic limit points were determined by the tensile yield of the surface plate. This phenomenon is the same as in the tests.





Shear Strength : $Q_{sc} = Q_s + Q_c$

$Q_s = t_s\{{}_1B_s \cdot \sigma_s(\varepsilon_s) + {}_2B_s \cdot \sigma_s(\varepsilon'_s)\}\cos 45°$

$Q_c = t_c\{(n+1) \cdot {}_1B_c \cdot \sigma_c(\varepsilon_c) + n \cdot {}_2B_c \cdot \sigma_c(\varepsilon_c) + {}_2B_c \cdot \sigma_c(\varepsilon'_c)\}\cos\theta$

Effective Width : ${}_1B_s = (h - \ell/4)\cos 45°$, ${}_2B_s = \frac{\ell}{4}\cos 45°$

${}_1B_c = {}_2B_c = \frac{h\sin\theta}{2(n+1)}$

$\theta = \tan^{-1}\{(n+1)\ell/h\}$, n : Number of Insert Plate

t_s, t_c : Thickness of Steel Plate and Concrete

σ_s, σ_c : Stress of Steel Plate and Concrete

$\varepsilon_s, \varepsilon_c$: Strain of Main Brace of Steel Plates, Main Brace and Inner Sub Broce of Concrete

$\varepsilon'_s, \varepsilon'_c$: Strain of Outer Sub Brace of Steel Plates and Concrete

Fig. 7 Analytical Model of Load-Displacement Relationship under Shear Load

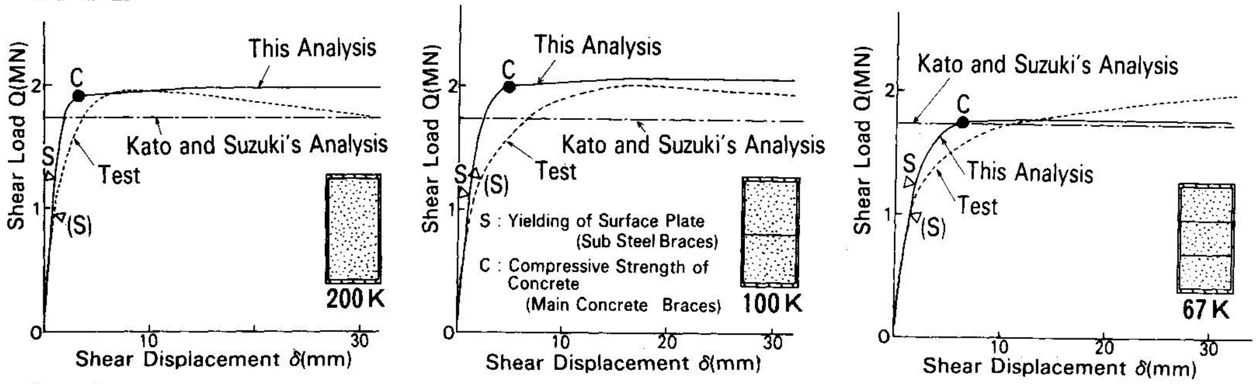


Fig. 8 Calculated Results of Load-Displacement Relationship under Shear Load

4. CONCLUSIONS

The thin steel plate SC bearing walls showed high strength and sufficient ductility due to the composite effect of the steel plate and the concrete. The buckling of the thin steel plate had little affect on the relationship between loads and displacements. The superposed values of the calculated steel strength and concrete strength is considered as a possible method to evaluate the strength of those structure. The calculated relationship between loads and displacements using a replaced brace model was in agreement with the test results.

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2. KATO, B and SUZUKI, H, Shear Strengths of Concrete Filled Steel Box Elements, Transactions of AIJ, No. 341, Feb. 1984, pp.27-36, (in Japanese).
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Poutre composite préfléchie précontrainte pour ponts et bâtiments

Composite Pre-Bent Prestressed Beam for Bridges and Buildings

Vorverformte, vorgespannte Verbundträger für den Brücken- und Hochbau

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Marie-Anne Belfroid, diplômée ingénieur civil des constructions à l'Université de Liège en 1958, a developpé dans l'usine familiale de grands éléments en béton précontraint. Elle a constitué une équipe de recherches qui a étudié et mis au point la réalisation d'ouvrages exceptionnels en éléments préfabriqués.

RÉSUMÉ
Il s'agit d'une poutre mixte capable de supporter de très fortes charges pour de très grandes portées avec une hauteur de construction minimum et pour un coût acceptable. Le principe en est l'utilisation d'une poutrelle en acier préfléchie et précontrainte qui est ensuite enrobée de béton lui-même précontraint par adhérence et éventuellement par câbles. La performance s'explique par l'exploitation optimale en tout point de chaque section des caractéristiques des composants – tous à haute résistance – et n'est possible que par la maîtrise du calcul, et facile que grâce à l'informatique.

SUMMARY
This composite beam can carry heavy loads over large spans at a minimum building height and at a very reasonable cost. The composite beam is based on the following principle : it is composed of a pre-bent prestressed beam coated with concrete that is itself either prestressed through bonding or post-tensioned, whilst in all section points properties of the high strength components are put to an optimal use. Mastering of the calculation procedure and the use of a computer are indispensable elements for the realisation of this performance.

ZUSAMMENFASSUNG
Es handelt sich um Verbundträger für die Aufnahme sehr grosser Lasten bei grossen Spannweiten und minimaler Bauhöhe. Der Träger setzt sich aus einem vorverformten und vorgespannten Stahlträger, welcher mit Beton umhüllt wird, zusammen. Der Beton wird dabei durch Haftspannungen oder auch nachträglich durch Kabel vorgespannt. Die hohe Tragfähigkeit ergibt sich durch die optimale Ausnutzung der hochfesten Baustoffe in jedem Querschnitt des Trägers. Dies ist nur möglich dank der Beherrschung der Festigkeitsberechnung. Rechnerunterstützte Berechnungsmethoden sind dabei unentbehrlich.

1. CONCEPTION

Les idées de base du nouveau procédé sont les suivantes :
- Utiliser une poutrelle métallique précambrée et précontrainte posée sur 2 appuis d'extrémité.
- Ramener la fibre inférieure en situation horizontale par préflexion, enrober la poutrelle de béton armé, la semelle inférieure étant pourvue de torons de précontrainte par adhérence.
- Relâcher la préflexion mettant la semelle inférieure en compression, à laquelle s'ajoute encore la précontrainte par adhérence.
- Pour garder des contraintes admissibles dans la semelle supérieure, prévoir une précontrainte provisoire agissant sur cette semelle.

2. DESCRIPTION DU PROCEDE

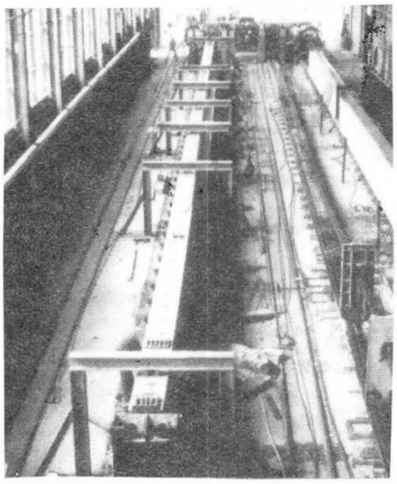

Fig.1 Poutrelle installée

2.1. Une poutrelle métallique est précambrée lors de sa fabrication par laminage ou par assemblage de plats soudés. Elle est sollicitée par préflexion et compression longitudinale excentrée de manière à éliminer les contraintes résiduelles de construction et à leur rendre un comportement élastique. La poutrelle est alors mise en place sur le fond de coffrage d'un banc de précontrainte à l'usine de préfabrication.
Des portiques métalliques la maintiennent dans la position voulue et empêchent tout déversement de la semelle supérieure. (fig.1)

2.2. Une précontrainte excentrée est appliquée à la semelle inférieure de la poutrelle sur tout ou partie de la longueur et vient augmenter la précambrure initiale en même temps que précomprimer la semelle inférieure.

2.3. Deux efforts verticaux sont appliqués à la poutrelle métallique et la mettent en préflexion, la ramenant dans une position horizontale.

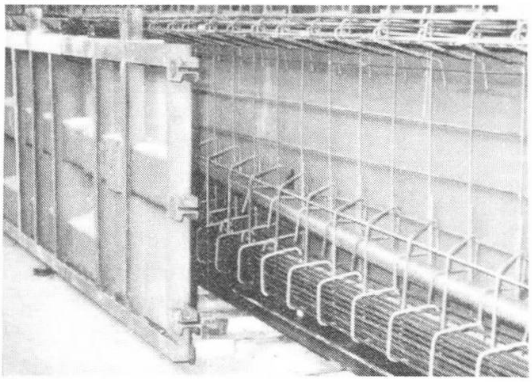

Fig.2 Poutre prête au bétonnage

2.4. Des armatures de précontrainte par adhérence du béton sont disposées autour de la semelle inférieure de la poutrelle et tendues entre les ancrages du banc. Des câbles de précontrainte à tendre sur chantier sont mis en place de même que les armatures complémentaires passives ou actives (armatures technologiques, étriers, armatures de renfort,...) (fig.2). Les coffrages latéraux sont fixés sur le fond et remplis de béton serré par vibration agissant sur les coffrages eux-mêmes. Le bétonnage est arrêté au niveau de la semelle supérieure de la poutrelle.

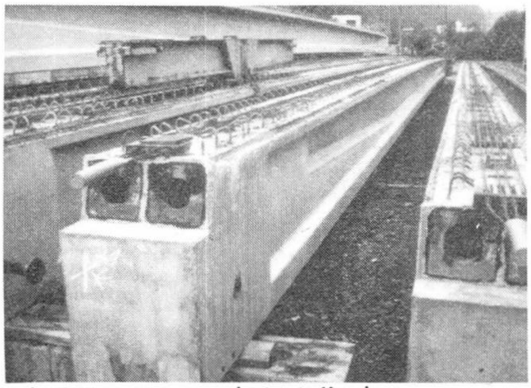

Fig.3 Poutre terminée à l'usine

2.5. Des armatures de précontrainte sont mises en place et tendues sur la semelle supérieure de la poutrelle restée apparente ("antiprécontrainte" à caractère provisoire).

2.6. Les efforts de préflexion sont relâchés mettant la poutre mixte en précontrainte et mobilisant simultanément son poids propre.

2.7. Les torons de précontrainte par adhérence sont relâchés apportant un complément de précontrainte à la poutre mixte. La poutre est mise au stock. (fig.3).

2.8. La poutre est transportée, mise en place sur chantier et reçoit le coffrage de la dalle, c'est-à-dire généralement des prédalles en béton.

2.9. Les efforts de précontrainte sur la semelle supérieure sont relâchés.

2.10. La dalle est coulée.

2.11. Les câbles de précontrainte chantier sont mis en tension et injectés.

2.12. La poutre est apte à reprendre les charges permanentes et les charges mobiles.

3. EVOLUTION DES CONTRAINTES (σ)

Nous allons suivre cette étude dans le cas précis du viaduc de Lixhe qui est un bon exemple d'application du procédé dans son ensemble.

3.1. Caractéristiques du viaduc (fig.4) :

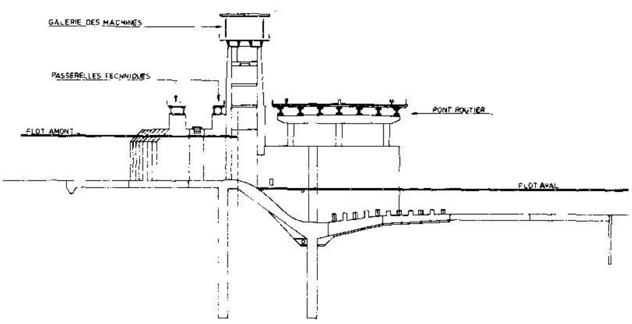

Fig.4 Coupe transversale de l'ouvrage.

Cet ouvrage permet le franchissement de la Meuse à 16 km en aval de Liège à hauteur d'un barrage, ce qui a conduit l'Administration à choisir une solution de tablier à poutres préfabriquées présentant une hauteur totale de 1m55. C'est un pont routier à 4 bandes de circulation qui doit permettre le passage de convois lourds de 240 tonnes.

Longueur totale : 324 m dont 4 travées de 47 m pour lesquelles les poutres faisant l'objet du présent exposé ont permis une solution économique avec une hauteur de 1m39 pour les poutres, soit 1/34 de la portée.

3.2. Caractéristiques de la poutre et de ses matériaux : (fig.5.)

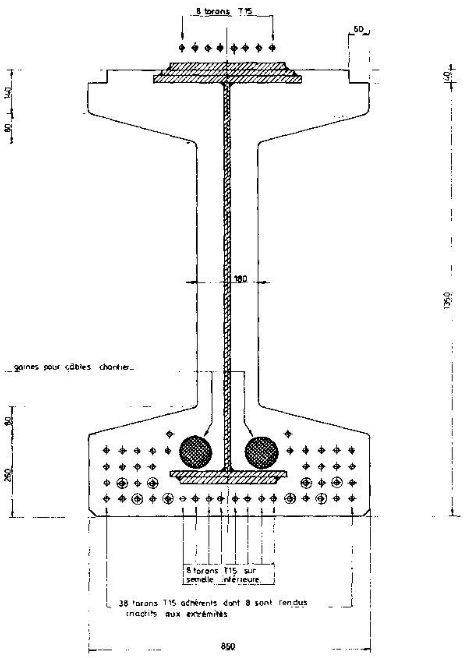

Fig.5 Coupe transversale de la poutre à mi-portée.

Longueur 46m90 - Portée 46m10 - Poids total 90 T

Poutrelle : Acier AE 355D - Ea = 215.000 N/mm^2
Tension max. service : 0,8 x 355 (= 288 N/mm^2)

T15.140.S : Charge de rupture 140 x 1,86 = 260,4 KN
Effort initial sur béton : 0,7 x 260,4 = 182,3 KN

Câbles 4T15.150 : Charge rupture 4x150x1,77=1062 KN
Effort initial sur béton : 0,7 x 1062 = 744 KN

Aciers BE 50 : Limite élastique 500 N/mm^2
Tension max. service : 0,6 x 500 (= 300 N/mm^2)

Béton poutre à la mise en précontrainte
Résistance moyenne R'b = 45 N/mm^2
Eb = 37000 N/mm^2 $\quad$ m_1 = Ea/Eb = 5,75

Béton poutre à 28 jours :
Résistance garantie R'wk = 54 N/mm^2
Eb = 42000 N/mm^2
m_2 = Ea/Eb = 5 (valeur instantanée)
m'_2 = 8,4 (valeur à la longue)

Béton dalle à 28 jours :
Résistance garantie R'wkd = 45 N/mm^2
Ebd = 37000 N/mm^2
m_3 = 5,75 (valeur instantanée)
m'_3 = 9,8 (valeur à la longue)

3.3. Calcul aux différentes phases d'exécution

3.3.1. Calcul élastique :

Les caractéristiques composées dans la section à mi-portée sont données au tableau 1.
Les distances des centres de gravité (G) sont données par rapport à la fibre inférieure .

Stade du procédé	Dénomination	Section m^2	Distance G m	Inertie m^4
[2.1]	Poutrelle métallique	0,056440	0,874	0,016313107
[2.2][2.3]	Poutrelle métallique précont.	0,057560	0,858	0,017058762
[2.5] à [2.7]	Poutre terminée au coupage(m_1)	0,152011	0,703	0,043032912
[2.6]à[2.10]	Poutre terminée à la longue(m'_2)	0,124576	0,721	0,036209845
[2.11]	Poutre+dalle à la longue(m'_2,m'_3)	0,180074	0,947	0,057152327
[2.12]	Idem+câbles+injection(m'_2,m'_3)	0,185718	0,925	0,060209387

Sollicitations propres à la poutre :
- Préflexion : 250 KN, entre-axe 18,60 M.
- Poids poutrelle métallique : 163,563 KN
- Poids béton+torons+armatures : 717 KN.

Poids de dalle et surcharges :
- Poids dalle + câbles + armat.: 12,68 KN/M.
- Charges permanentes : 8,95 KN/M.
- Charges mobiles : 26,97 KN/M.

Tableau 2 des contraintes en N/mm^2 dans la section à mi-portée : + = tract. - = compres.

Stade	Dénomination	Moment KN x M	Fib.inf acier	Fib.s. acier	Fib.sup béton	Fib.inf béton	Fib.sup dalle
[2.1]	Poids Poutrelle acier	+ 1057,26	+ 50,1	- 35,1			
[2.2]	Précontrainte poutrelle	- 1201,02	- 82,8	+ 14,0			
[2.3]	Préflexion poutrelle	+ 4650,00	+ 220,5	-154,3			
	Poutrelle avant bétonnage.		**+ 187,8**	**-175,4**			
[2.5]	"Antiprécontrainte" (ml)	+ 1730,79	+ 14,5	- 38,4	+ 3,23	- 6,50	
[2.6]	Lâchage préflexion (ml)	- 4650,00	- 65,2	+ 76,9	-13,02	+13,00	
	Poids propre béton (ml)	+ 3551,00	+ 49,8	- 58,7	+10,10	- 9,93	
[2.7]	Précontrainte adhér.(ml)	- 4773,53	-100,4	+ 19,1	-19,04	+ 2,90	
	Poutre terminée au coupage		**+ 86,5**	**-176,5**	**-18,93**	**- 0,53**	
[2.8]	Coffrage dalle (25% dalle)	+ 842	+ 14,4	- 16,2	+ 2	- 1,85	
[2.9]	Enlèvement antiprécontr.	-1730,79	- 14,5	+ 38,4	- 3,23	+ 6,50	
	Après coffrage dalle,pas pertes		**+ 86,4**	**+154,3**	**-20,16**	**+ 4,12**	
Valeurs à la longue							
[2.6]	Lâchage préflexion (m'2)	-4650	- 79,7	+ 89,2	-11	+10,2	
	Poids propre béton (m'2)	+3551	+ 60,9	- 68,1	+ 8,4	- 7,8	
[2.7]	Précontrainte (m'2)	-4773,53	-124,6	+ 21,6	-16,16	+ 2,24	
[2.10]	Coulée de la dalle (m'2)	+3367,49	+ 57,7	- 64,6	+ 7,98	- 7,41	
[2.11]	Précontr. chantier (m'3)	-4170	- 92,8	+ 3,1	-11,92	+ 0,16	+ 1,62
	Pertes précontrainte 10%	+1076	+ 22,3	- 1,2	+ 2,86	- 0,09	- 0,45
	Situation tablier terminé		**+ 31,6**	**-195,4**	**- 19,84**	**- 2,70**	**+ 1,17**
[2.12]	Surcharges fixes (m'2,m'3)	+ 2538,65	+ 34,8	- 20,7	+ 4,64	- 2,34	- 2,86
	Surch. mobiles (m'2,m'3)	+ 7163,96	+ 98,1	- 58,3	+13,10	- 6,59	- 8,08
	Poutre sous total charges		**+164,5**	**-274,4**	**- 2,10**	**- 11,63**	**- 9,77**

Quelles constatations peut-on faire à ce moment de l'étude ?

- La poutre est maintenue au cours des différentes opérations dans une situation de contraintes tout à fait confortable tant pour le béton que pour l'acier.
- Grâce à " l'antiprécontrainte " notamment, la poutre se trouve en compression à la fibre supérieure, ce qui permettra de lui donner certains porte-à-faux pour faciliter les manutentions de mise en place.
- Ce calcul réalisé dans toutes les sections de la poutre permet de connaître parfaitement la situation et donc d'exploiter au mieux les matériaux de la poutre composite.

3.3.2. Déformations

Le calcul des flèches aux différents stades du procédé et dans des sections entre-distantes de 1 m à partir de la section à mi-portée a été effectué. Nous résumons au tableau 4 les différentes valeurs dans la section à mi-portée.

Stade du procédé	Dénomination	Flèche : + = flèche - = contreflèche
[2.1][2.3]	Préflexion + poids propre poutrelle	+ 388 mm.
[2.2]	Précontrainte poutrelle	- 96 mm.
	Précontrainte poutrelle avant enrobage	+ 292 mm. fibre inf. acier
[2.6]	Relâchement préflexion	- 139 mm.
	Poids propre béton	+ 119 mm.
[2.7]	Précontrainte par adhérence	- 117 mm.
	Poutre complète au coupage	- 137 mm. fibre inf. béton
[2.6]	Relâchement préflexion	- 167 mm.
	Poids propre béton	+ 142 mm.
[2.7]	Précontrainte par adhérence	- 144 mm.
	Pertes de précontrainte	+ 20 mm.
	Poutre complète à la longue après pertes	**- 149 mm.**
[2.10]	Poids dalle à la longue	+ 104 mm.
[2.11]	Précontr. par câbles à la longue après pertes	- 66 mm.
	Tablier avant surcharges	**- 111 mm.**
[2.12]	Charges permanentes	+ 45 mm.
	Tablier sous charges permanentes	**- 66 mm.**
[2.12]	Charges mobiles (m_2 m_3)	+ 92 mm.
	Tablier sous charges mobiles	**+ 26 mm.**

Tab.3. Déformations.

3.3.3. Elastification de la poutrelle

Avant la mise en oeuvre de la poutrelle, il y a lieu de s'assurer de son comportement élastique et donc d'éliminer les contraintes résiduelles de fabrication. Pour ce faire, on soumet la poutrelle à des sollicitations qui créent en tous points des contraintes égales ou supérieures aux contraintes de service.
Pour ce faire, 2 poutrelles sont couchées sur des appuis à rouleaux, leur semelle inférieure mise au centre. Elles sont soumises :
- à flexion par tiges Dywidag agissant horizontalement pour les rapprocher.
- à un complément de compression introduit dans la semelle supérieure par une précontrainte à l'aide des torons " d'antiprécontrainte ". Ce complément est indispensable en raison de la dissymétrie de la poutrelle même et de l'importance des contraintes nécessaires en semelle supérieure.

Plusieurs mises en charge successives peuvent être faites jusqu'à obtention de flèches égales. En pratique, une seule opération a été nécessaire dans le cas présent.

3.4. Calcul à l'état limite ultime

Ce calcul a été fait suivant les méthodes du CEB en adoptant les coefficients de sécurité repris au tableau 4.

Dénomination	Béton poutre	Béton dalle	Barres BE50	Poutrelle	Précon-trainte	Charges permanentes	Charges mobiles
Coefficient	1,4	1,5	1,15	1,15	1,15	1,35	1,5

Tab.4. Coefficients de sécurité

On obtient : Moment Résitant/Moment sollicitant = 26482 KNm/24941 KNm = 1,062

3.5. Constations et mesures effectuées

3.5.1. Contrôles de contraintes et déformations : La fabrication des 28 poutres décrites ci-dessus a fait l'objet d'un autocontrôle de l'usine suivi journellement par la 2ème Division du Bureau des Ponts à Liège qui a par ailleurs établi un programme de mesures permettant de connaître le mieux possible les déformations et contraintes ( ) à mi-portée et de les comparer aux valeurs calculées. Pour ce faire, 9 jauges de contraintes et 2 extensiomètres à corde vibrante ont été placés en différents points de la section médiane d'une poutrelle. Les différentes mesures sont reprises au Tableau 5.

Stade procédé	Dénomination	σ calcul.	σ mesurées (N/mm^2)		Fibre
[2.1]	Elastification	- 244 + 320	- 233 + 278		Sup. Inf.
[2.1] à [2,3]	Poutrelle avant bétonnage	- 175,4 + 187,8	- 172 + 171	- 168 (après 7 J) + 170 (après 7 J)	Sup. Inf.
[2.5] à [2,7]	Poutre au coupage et évolution	- 176,5 + 86,6	- 157 + 28	- 153 (à 21 J) -151 (à 132j) + 66 (à 21 J) + 30 (à 132j)	Sup. Inf.
[2.8] [2.9]	Dalle coffrée sans antipréc.	- 154,2 + 86,5	- 149 + 10	(sans pertes)	Sup. Inf.
[2.10] [2.11]	Dalle bétonnée et précontr.chantier	- 195,4 + 31,6	- 205 + 43	(avec pertes)	Sup. Inf.

Tab.5 Comparaison des contraintes poutrelles calculées et mesurées.

La corrélation entre valeurs théoriques et valeurs mesurées est satisfaisante. Toutefois, on peut se demander quelles valeurs de m conduiraient à une concordance encore meilleure. On trouve : au coupage des torons adhérents, m = 8,5 ; à 21 jours , m = 13,5 ; à 132 jours, m = 18. Néanmoins, l'utilisation de semblables valeurs pour le calcul correspondant des contraintes est tout à fait aberrante et nous en avons, bien involontairement eu la preuve. Lors du transport par eau des poutres vers leur destination, un chargement a accidentellement basculé et des poutres se sont retrouvées au fond de la Meuse dans une position quelconque. Elles en ont été retirées de telle sorte que les contraintes en cours de manutention auraient atteint plus de 25 N/mm^2 dans certaines sections qui n'ont présenté aucune dégradation importante. Il y a donc bien eu une redistribution des contraintes entre béton et acier, l'acier ayant empêché le béton de prendre des allongements correspondant aux contraintes théoriques. Sans doute, faudrait-il trouver un moyen dans les calculs de plafonner la résistance du béton à la traction tout comme cela se fait à la compression dans les calculs aux états limites.

3.5.2. Mise en charge d'une poutre avant sortie d'usine : Un essai de mise en charge a été réalisé 21 jours après relâchement de la précontrainte par adhérence par application de 2 efforts de 250 KN à 4m45 de l'axe de la poutre (M = 4650 KNm). Les mesures de contraintes dans la poutrelle et les flèches élastiques mesurées correspondaient à des valeurs calculées pour $5 < m < 6$.

4. CONCLUSION ET COMPARAISON ECONOMIQUE : Les mesures effectuées confirment la fiabilité des méthodes et coefficients de calcul utilisés ainsi que des méthodes de fabrication. Economiquement, la comparaison entre la poutre nouvelle et ses consoeurs, poutre précontrainte classique et poutre préfléchie classique se résume comme indiqué ci-dessous à égalité de portée, charge et hauteur de construction.

	Poutre nouvelle	Poutre préfléchie	Poutre précontrainte
Nombre	7	7	15
Poids poutrelle	114,5 T	255,7 T	-
BE50	20,1 T	22,6 T	59,6 T
T15, 140S	18,1 T	-	30,3 T
T15, 150	11,4 T	-	28,8 T
m^3 béton	201,3 m^3	282,7 m^3	590,2 m^3
Rapport prix placé	1	1,60	1,45

Axial Shear Connectors for Wide Span Prefabricated Structures

Goujons axiaux pour structures préfabriquées de grande portée

Axiale Schubdübel für vorfabrizierte Balken mit grosser Spannweite

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SUMMARY
The purpose of achieving spans for bridges of up to 90 meters by means of prestressed simply supported structures adopting light composite truss beams supporting cast in situ floor decks is getting nearer and of real economical interest. Coupling of steel diagonals and struts to prefabricated prestressed concrete chords is facilitated by flexible-type shear connectors. The paper describes tests carried out during ten years and first realizations by the author.

RÉSUMÉ
Le franchissement de portées de 90 m, par des poutres librement appuyées, précontraintes, est devenu intéressant par l'utilisation de poutres-treillis légères en construction mixte acier-béton précontraint, soutenant des prédalles complétées in situ. Le couplage entre les barres métalliques du treillis et les éléments préfabriqués est obtenu au moyen de goujons flexibles. L'étude donne une description des expériences conduites pendant dix ans par l'auteur, et les premières réalisations.

ZUSAMMENFASSUNG
Die Möglichkeit Brückenspannweiten bis 90 m mit Hilfe frei aufgelagerter, vorgespannter Balken zu erreichen kommt näher und wird wirtschaftlich interessant, wenn vorgespannte Fachwerk-Verbundträger verwendet werden, welche mit einem an Ort gegossenen Ueberbeton zu ergänzen sind. Die Verbindung der Fachwerkstäbe aus Stahl mit den vorfabrizierten Betonelementen wird mit Hilfe flexibler Schubdübel erreicht. Der Beitrag beschreibt Versuchsergebnisse und erste Anwendungen.

1. FOREWORD

A careful investigation on shear connectors particularly fit to prefabricated structures and aimed at reaching wide spans, using composite truss beams completed by cast-in-situ floor decks, was conducted by the author during ten years. First realizations in Italy, ranging from highway bridges to floor decks for nuclear plants, display an ingrowing interest for this type of structural solution.

2. EXPERIMENTAL INVESTIGATION

2.1. Statical Tests (Asymmetric Specimens)

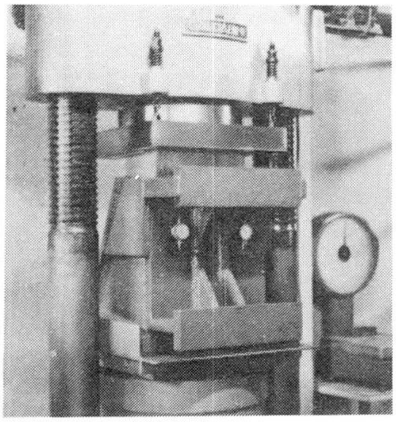


Fig.1 Tests on asymmetrical shear connector specimens

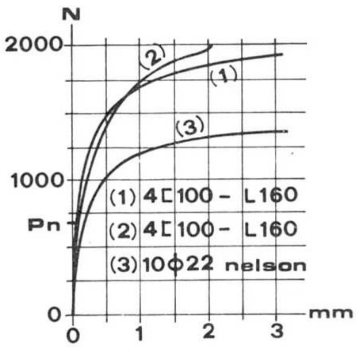


Fig.2 Relative displacement vs. load for different types of shear connectors

Early tests at the Laboratory of the Istituto di Scienza delle Costruzioni at Turin Polytecnicum during 1978 on four shear connectors specimens (2 specimens with channel and 2 specimens with Nelson headed stud connectors) persuaded the author to choose the light channel type connector, so much that he stopped testing the forth specimen equipped with Nelson-type connectors (curve three of Fig.2). Intensive and more carefully conducted tests by other authors (1),(2),(3), explain in detail the soft behaviour of the Nelson type,(4) compared with the rigid behaviour of the channel-type connector (curves 1 and 2 of Fig.2).

A second serial of tests was conducted in 1979 at the same Laboratory of Turin Polytechnicum upon 2 real scale specimens of truss beams spanning 6.36 meters (5). Location of the 149 measurement bases on two beams are shown in Fig.3 and the quite low relative displacements between steel plates and concrete are illustrated in Fig. 4, giving good accordance with early results.

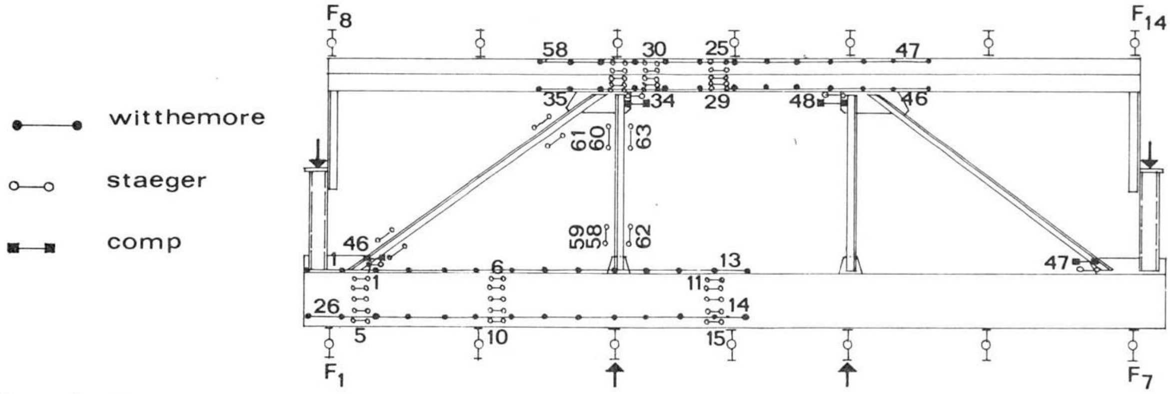


Fig. 3 Measurement location for the 2 beams tested at Turin Polytechnicum

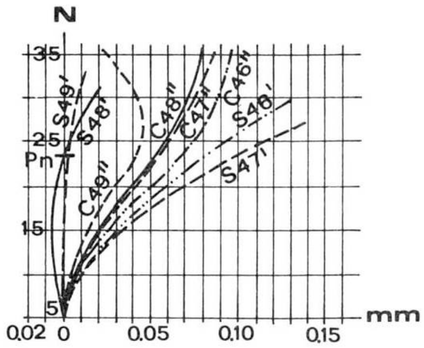


Fig. 4 Relative displ./load for Fig.3 beams

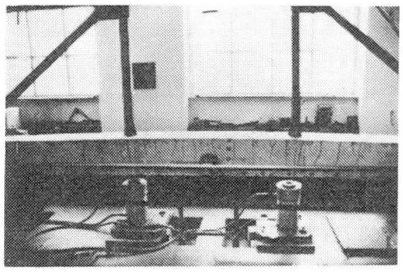


Fig.5 The limit state for one of the tested beams of Fig.3

Both beams were tested by means of two hydraulic jacks of 1000 kN and behaved almost elastically up to working conditions; the second beam (Fig.5) showed a permanent deflection of about 0,20 mt. after a loading range totalling 2.3 times the working load; the whole beam revealed full exploitation of all structural elements (p.c. and r.c. chords, steel diagonals and sruts, shear connectors and gusset plates).

2.2. Tests - (Symmetric Specimens)

Push-out tests on 3 double specimens (A Serie) equipped with (80x45x6)mm. channels and measuring (160x120x200)mm. (dimensions are slightly different from the standard AIPC-CEB-CECM-FIP ones) in 1983 at Milan Polytechnicum, revealed the intrinsec sensitivity of the proof to arch effects related to lateral thrust at the base of specimens. The third test with reduced distance for the two lower hinges, showed however lower rotations of twin specimens up to ultimate load, and an almost tangential working for the shear connectors in the elastic range: this effect caracterizes axial shear connection for truss beams.

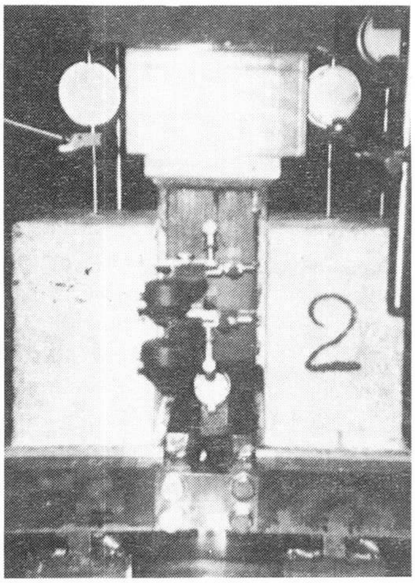


Fig. 6 Statical tests of symmetric specimens

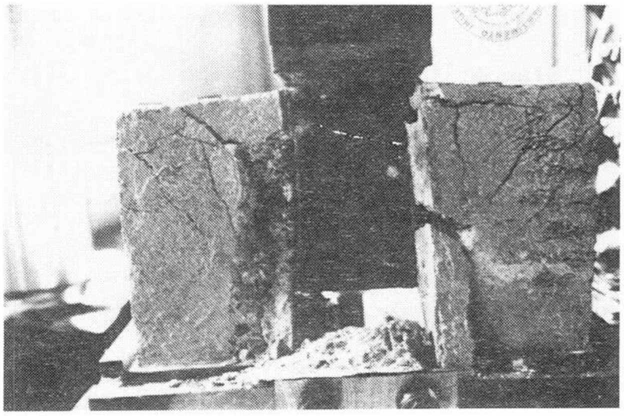


Fig. 7 Failure of specimen B5 under static load

2.3. Dynamic Tests - (Symmetric Specimens)

Dynamic pulsating tests on 7 double specimens (B Serie) same as the static tested ones, were carried out also in 1983 at Milan Polytechnicum. The pulsating force was held between 40 and 150 kN, by a frequence of 9.9 Hz. Each sample underwent 2.E6 loading cicles without visible cracking effects. Seven stops during tests made it possible to perform a statical test under a load of about 40 kN. The measurements showed a longitudinal adaptation less than 0,1 mm. of the tested specimens for the first 10000 cicles, slightly increasing for 2.E6 cycles, and an angular rotation of the head of the specimen held constant. Another result of the tests, which will be illustrated more in detail in a next publication, was the tendance of tested specimens to raise internal states of self-stressing related to cracking, under localized shrinkage effects. Three specimens were also statically tested up to failure as shown in Fig. 8 averaging an ultimate load not lower than that reached by the same specimens not prooved by dinamic tests. Failure was given by crashing of concrete, and almost simultaneously reached excessive elongation of connectors web. Fig.7 shows a characteristic failure crack pattern for specimen B2 which permits to "read" the real path of compressive struts through concrete. The steel connector was a St 430 MPa, weld was magnetically inspected, and a preliminar chemical investigation was made for mill steel. Concrete reached a characteristic R'bk = 50MPa cubic strength at 28 days.

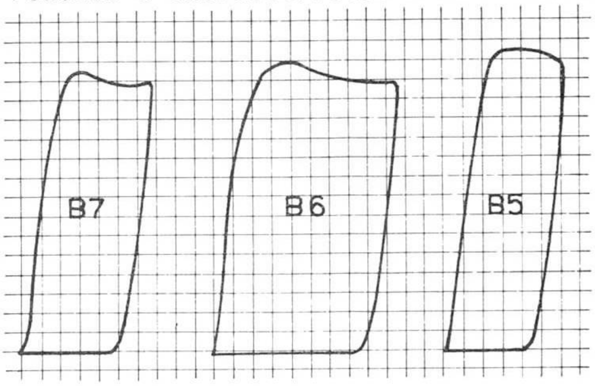


Fig. 8 F-ε diagram for static loading to failure, of 3 dynamic tested specimens.

3. APPROACH WITH F. E. METHOD.

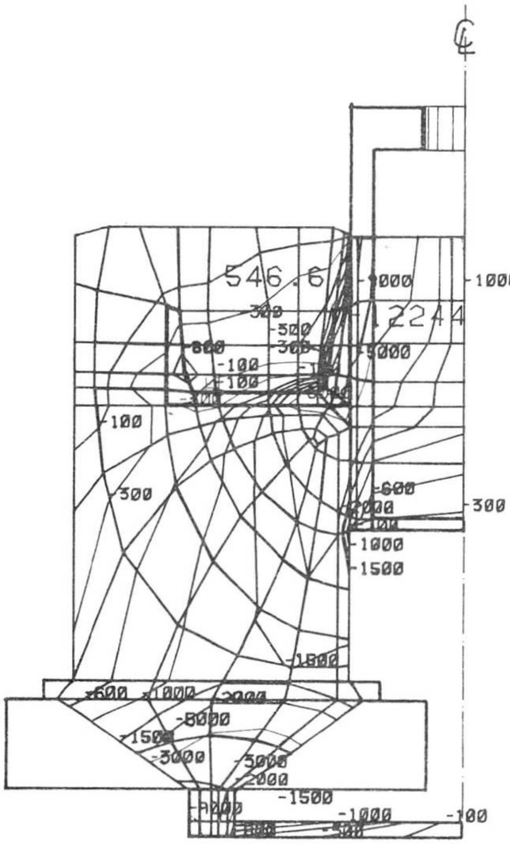


Fig. 9 Stress level plot after elastic analysis with a F.E.M. mesh of 151/2-D elements.

An analysis conducted with 2-D quadrilateral strain F.E. confirmed high stress concentrations at the fore toe of the channel;after inibition of some F.E. at the interface with the steel pipe. Though quite inadequate, the elastic analysis showed that real behaviour of this type of connection is far from being well understood and needs for more intensive and widespread ascertainement in the future. This gap of our knowledge has been recently remembered by other investigators (6).

4. UP TO DAY REALIZATIONS.

The first realizations include 2 bridges over the river Melezzo built during 1982 in the high Vigezzo Valley, about 30 Km. far from Locarno. The Mugnago bridge spans 3x40 and the Meis bridge 30 -45- 30 meters. Both are pedestrian bridges designed for a 270 kN truck. The bridge deck was entirely prefabricated in a plant 300km. away, and assembled within 2,5 months/ bridge. Each 1/3 span single beam segment was assembled during a workday by 4 workmen,and raised on provisional piers by a little self-moving crane: maximum load of one segment was 170 kN. Two days after sealing of 60 mm.thick chord joints by means of high strength cement mortar; tensioning of the (2x2) lower chord 1200 kN/each prestressing tendons produced self-raising of both main beams. Another group of realizations still under construction comprises the aseismic floor decks of 4 major buildings of the nuclear plant ENEL (general designer ANSALDO) near Rome. The flat deck of the Turbina building for ex. measures

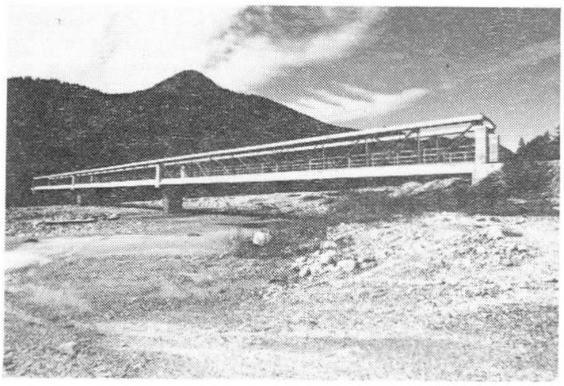


Fig.10 The Mugnago Bridge spanning (3 x 40,0 meters)

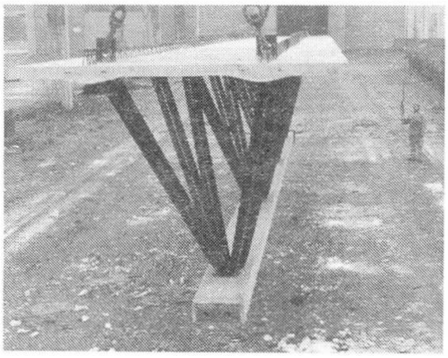
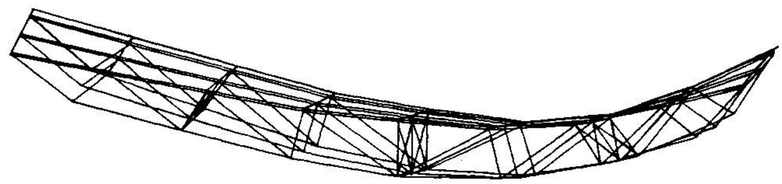


Fig. 11 Assembler beam for nuclear plant near Rome

(115,0x35,0)m. and is a completely isolated plate 0,40 m. thick, supported by 48 adjacent Assembler-type truss beams and post-tensioned by 46 cables; linked to the exterior wall of the 35,0 m. high structure by means of some 38 hydraulic shock-absorbers and 4 neoprene/teflon bearings of 6000 kN (7). The last realization is the Plusc bridge of Verbania on the boarder of the Maggiore Lake in North Italy, spanning 70,0 m., also under construction. The bridge deck is designed for highway loads and is supported by 3 Assembler-type beams weighing 2100 kN spaced at 3,75m.

Fig. 12 Elastic deformed structure of the 70,0 m. span of Plusc Bridge of Verbania



5. THE ECONOMICAL SIDE.

Computations of designed works give (Table I) the following values for building materials. Table II gives extrapolated items for a 90 m. span.

Concrete of the plate-deck	0,24 m^3/m^2	0,24 m^3/m^2
Concrete of pref. beams	0,32 m^3/m^2	0,65 m^3/m^2
Structural steel	1,05 kN/m^2	1,33 kN/m^2 *
Prestressing steel	0,17 kN/m^2	0,34 kN/m^2
High-bond steel pref. beams	0,19 kN/m^2	0,40 kN/m^2
Table I Materials for a 70,0m. h.-bridge		Table II Materials for a 90,0 m. h.-bridge

* adopting for diagonals and struts circular seamless pipes.

Further savings will be obtained by utilization of high strength concrete so to reduce the weight of the prefabricated truss beams.

6. CONCLUDING REMARKS.

This new type of structure permits to improve the range of spans for prefabricated structures, eliminating the heavy webs of traditional prestressed beams. Contemporary use of prestressing steel and structural steel for the beams, and r.c. in the upper zone of the deck, avoids provisional windbracing of steel beams during erection and gives best behaviour under impact and dynamic loading.

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