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Concepts of Composite Construction - Mutatis Mutandis

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Summary

Composite construction encourages a focus on minutiae which becomes self reinforcing. Composite construction can be seen as a system concept. If one reviews it on that basis, the huge scope awaiting exploitation reveals itself.

Introduction

Construction and in particular structural design seem to stimulate the desire to extend the scope of what can be analysed holistically or in a unified manner, what we might describe as the analysable entity. In civil and structural engineering, this desire has often been focused on construction made of a notionally single material, reinforced concrete or stone or steel.

The development of our profession is illuminated by a series of distinct redefinitions of that analysable entity. Some become simply bigger or more complex, some arise from advances in technique, some are by the articulation of elements to disaggregate the difficult whole and some focus on conceptions of the technologies and processes which bring our structures into being.

Examples of the bigger or more complex include many of those bridges which are the proud record of IABSE members. But they also include structures more modest in scale, such as the shells of Candela or the membrane structures of Frei Otto or the glass structures of Tim MacFarlane.

Examples of advances in technique include the development of limit state theory, matrix methods, finite element analysis and Jacques Heyman's analyses of Gothic stone structures.

The Forth Railway Bridge and the Buckminster Fuller "tensegrity" structures both demonstrate the power of articulation into tension and compression elements. The structural behaviour, the system points and load paths, can be sensed and read off the completed structure.

In some cases the actual articulation is used to dissect the analysable entity out from its complex surroundings. A recent case is seismic design work for a Californian building making use of base isolation elements to separate the upper structure, which is then regarded as a virtual pendulum.

The intellectual restatement of the general process with which I am most familiar is what is called total architecture or total design, as formulated by Ove Arup. This conception has itself been interpreted and developed in use, with our growing understanding of the detailed implications of design. It is always characterised by the creative tensions between synthesis and analysis, between harmony and invention, between the established and the unknown and hopefully by sufficient eventual reconciliation.

We should pair the topic of the analysable entity with the framework of time which gives us the order to events that we call history. Civil engineering structures have usually been conceived as one act, however large and even if their actual construction stretches over decades.

With building structures, however, it is a surprisingly recent phenomenon, probably less than 300 years old for major buildings again to be designed as a unity and then to be completed more or less without change. Sir Christopher Wren was the first modern designer of a cathedral to see his design built. Brunelleschi's Duomo was constructed on supports which had been designed and built more than a century earlier and with little idea of how the space could be spanned. Many may argue bitterly that we have again regressed and design in parts as the construction proceeds.

Time is significant in three additional ways:

- we seek durability and longevity of predicted performance
- we add elements for changes of use and we add repairing or strengthening elements to cope with wear and ageing
- we use temporary works and falsework as a crucial element in the process or explore whether to eliminate them. The design for the Kingsgate Footbridge in Durham is an example of treating the temporary works by rotating the structures as integral to the total concept.

Composite construction

There is nothing unusually special about composite construction. The composite concept is very old.

The origins of the minerals and metals which we use in construction are in the ground in an unhelpful mix of composites. We variously recover, refine, transform and recombine these minerals into our composite materials, cements, plasters, concrete, brick, clay tiles, terra cotta, irons and steels, aluminium alloys and glasses.

The natural organic building materials (straw, reeds, timber and so on) are in themselves, composite materials. Natural structures offer us fascinating models for structural form and environmental control through their material arrangements.

One of man's earliest deliberately composite materials is probably the sun-dried, straw-bound brick. Even then, there was some understanding of the criticality of quality control, the history recording the warning about bricks without straw.

The use of iron cramps in the Acropolis stonework, of Victorian cast and wrought iron and of timber roof trusses, right through to today's use of steel/concrete and advanced polymers, all provide more recent examples of the use of the composite technique.

Today, we have started to combine materials to exploit and extend our modern understanding of composites; steel with concrete, glass and carbon with polymer and so on.

Composite construction thus describes the combination of elements or materials in ways which can be regarded as delivering a single analysable entity. These have also meant the synthesis of discrete elements, extending the spatial extent of that analysable entity. The extraordinary bridges by Maillart show us how we can extend the spatial scope of what we learn as elements into one whole bridge. The work of Fazlar Khan gave the language for a major evolution in our structural concepts of the tall building. A less obvious example is the long history of developing the structural theory of the column.

The Pegasus Paradigm

Across the axis of time we have in composite construction, a further dimension, that of definition through use and familiarity.

The creature Pegasus was formed by combining the body of a horse with the wings of a bird. Initially we comprehend the idea through the properties and qualities which the elements of composition, the horse and the bird bring to the whole. Later, ***through use*** and in language, narrative and recollection we come to treat the overall idea as a concept in itself and to realise what is distinct and gives new meaning in the unified concept.

We find this in design. A motor-cycle is more than an engine plus a bike and has become a distinct single concept. We know that e-mail is more than electrified correspondence.

So it is with composite construction. The most familiar example is reinforced concrete, which so often we can usefully regard as homogenous, as a single isotropic material. The RC paradigm has some of the original properties of the separate constituents but more importantly it has its own qualities not possessed by any one constituent.

The rediscovery today of lime putty mortars adds to our options of cement mortars, from which we see more clearly that bricks-and-mortar is a wide repertory of composite materials. This counterpoint of new and old brickwork as distinctive materials is nicely demonstrated in two buildings designed by Michael Hopkins. Glyndebourne Opera House has lime putty mortar to eliminate movement joints. The Inland Revenue Centre has cement mortar, so that the brickwork could be built in the factory and then be transported and erected on site as precast elements.

In the vista of composite construction, the most interesting issues are those made possible by the new understandings and new possibilities of the composites, those aspects which are not properties of the separate parts. These are our contribution to the Pegasus paradigm.

The Challenge of Composites

We are at the stage in the development of composite construction where we can ask some questions

- How do we systematise composite materials, composite structures and composite construction so that we will discover new possibilities of form, geometry, connection, detail and performance as the norm of our construction process?

- How do we apply composite ideology? Have these ideas transformed our thinking?

- How will doubters come to permit the use of apparently untried novelties, such as are the inevitable progeny of the composite approach? Can we evolve to a concept of controlled innovation which is customary and reliable?

We now understand established and potential construction materials in fundamentally new ways, because of innovations in knowledge, interpretation and measurement. These have led to improved knowledge and understanding of

- Materials, where investigative techniques now permit a molecular level of understanding, relating this to macroscopic engineering properties.

  In some cases, this has enabled us to rediscover some traditional materials and techniques, tailoring combinations of these materials to demand. The modern developments of ferrous castings use 19th Century craft increasingly combined with the new understandings gained through computer simulations of a casting's cooling behaviour and through fracture mechanics. These have been used in a progression of building projects: Bush Lane House, London; Centre Pompidou, Paris; Alban Gate, London; Bracken House, London; Ponds Forge International Pool, Sheffield; Menil Gallery, USA; Western Morning News, Plymouth.

  In future, we will be able to tailor materials to meet requirements, whether of the process or for the final service in place. We can regard our principal structural materials concrete, steel, masonry, timber and polymers as each referring to families of materials, creating in our minds a more continuous spectrum.

- Structure, where advanced computer methods make complex analysis freely accessible through modelling, analysis of elements and inter-action of elements. These allow a unified view of many structures which hitherto had to be analysed and hence handled in stages.

- Construction or organisation of production, where advanced manufacturing techniques like CAD-CAM can create a direct link from design simulations to production information. This will also result in higher levels of achievable and therefore demanded precision with all that follows for the quality in our product.

- The technological and industrial context, where the construction industry hovers uncomfortably at the gate still unsure whether to change radically the concept of the construction process.

We find that we are able to design composite constructions and structures which are significantly larger and more complex in space, time and material. They are a larger proportion, sometimes almost 100%, of the whole construction. We have substantially extended and redefined the scope and practical meaning of the analysable entity. Our total engineering is increasingly coterminous with total architecture.

This review is important because of changes in the context of the development of the engineer's work

- we should seek to confront with all our energy the emerging social demands of the huge mega-cities of the 21st Century, built on and in unmeasured terrain and environments, in desperately poor but irrepressibly optimistic congregations of citizens.

- we are able to contemplate greater scope of admissibility of technical solutions and methods, because of our increased ability to control their behaviour, even their meaning.

- we will soon be faced with the puzzle of how we decide structure and its form, when almost any material can be tailored to suit our process or performance requirements and almost any form can be analysed, the classic problem of rich choice.

  In particular, we now have sufficiently powerful methods and understanding to be able to consider the behaviour of an extraordinary range of different combinations of elements and structures by definition extending beyond the scope of codes of practice.

Can any engineer resist this fascinating prospect?

Meeting the challenge

One of the most pleasurable functions of the designer is to define and promote good overall ideas which the client and users had not realised were even possibilities. Solutions of this kind are typically creative adaptations of previous solutions and methods from other projects or industries. Fundamentally new solutions are extremely rare.

Recognition of what constitutes composite construction allows us to contemplate a much larger set of potential responses to existing problems, opportunities and ambitions along with an extension of the possibilities in confronting new situations altogether.

Composite materials

The construction sector is characterised by a long list of performance requirements that must all be met in some measure.

In terms of materials, it is currently possible to select a large range of materials of similar properties or performance for a particular application. Such freedom is available because of the controllable or specifiable versatility of current materials. Metals can vary their properties by alloying with different metal or fillers, concretes can vary their properties by selection of different aggregates or reinforcement whilst polymer composites may vary their properties through selection of different matrices, fillers or fibres.

The construction materials industry has produced this huge spectrum of materials on the macro scale. By altering the combination of different materials, almost the full range of properties can theoretically be obtained within a composite whole framework. It is becoming possible to create a range of advanced material solutions tailored to almost any design requirement. Many will become viable on cost as well. It is usually possible to identify the straightforward material solution; the challenge for inventive designers is to develop real benefits for their clients and users through consideration of a many new material developments or new combinations of existing materials.

Created needs are a common concept in other industries. Personal stereos (Sony Walkman) and home video games came more from the supply side of a design technology push perspective than from the demand side of consumer pull.

It maybe difficult for a bridge designer to develop new concepts without considerable support from clients and investors to reach the required confidence. Nonetheless, such an approach has been part of our history. It is an essential approach if our industry is to innovate and best make use of the opportunities that composite materials and systems provide.

For architects, this approach can be realised with relative ease since they are largely relieved of the realities of delivery carried by the engineers and builders. Richard Rogers' concept for the Centre Pompidou with Renzo Piano and the Lloyds' Building concentrated on total flexibility of use within a concept of heavily populated space and its exploitation. The buildings themselves grew systematically from this idea, adapting to the need for services, fire protection, access and the like. The central focus, of total physical re-arrangeability for users, was substantially achieved in the actual constructions.

Technically, the design of the Barcelona Communications Tower was driven by the need to deliver radio transparency. By recognising advantages of non-metallic tendons at an early stage, the designers were able to achieve sufficient confidence in the design of this new material to beneficially exploit the use of these materials in the final structure.

From specific innovative solutions, further innovation can be released to benefit subsequent projects. The confidence gained from the use of non-metallic tendons at Barcelona was a key factor in providing justification for the use of composite non-metallic prestressing in a concrete reservoir in Nottingham. Whilst the prime motivation for use differs (radio transparency or long-term corrosion resistance) the innovation continues from one application to the next. This is achieved partly by publication but more usefully through the personal confidence of the participants. Without practice innovation can rapidly cease. It is therefore strategically important to society that a culture of innovation be sanctioned.

Composite structures

Cases to illustrate possibilities in composite structures for new designs are the Commerzbank HQ structure in Frankfurt and the use of the New Austrian Tunnelling Method.

Much work in the USA, Japan and elsewhere has focused on the big issue of remedial work for old infrastructure, where the compositeness arises through the use of new composite materials per se and their use in being added to existing materials or structures. A USA survey identified the following priorities

- corrosion mitigation, by replacing or protecting metallics with composite elements.
- strengthening degraded bridge components, where composites can replace metallic or concrete elements which are subject to characterised stress or degradation.

- seismic retrofit, by jacketing under-designed elements.
- transforming into very low-cost erection or low-cost maintenance structures, such as pedestrian bridges, maintenance walkways.

This potential for enhancing existing structures demonstrates the further extension of composite, the combination of the old and its repair, a time-dependent case of the composite analysable entity.

Organisational and production issues

In some projects, we have seen the power of a production perspective in developing structural design.

Peter Rice exploited the potential from resolving a structural form into repetitive elements which would rationalise and economise production as well as expressing a powerful overall design theme. He sometimes combined this with an attempt to develop composite structures in which the constituents' weaknesses were deliberately confronted and resolved in the Pegasus mode. Examples include the IBM travelling pavilion and the Seville pavilion. His critical emphasis was the pursuit of regularity, repetition and modularity, of standardisation in the best sense, thereby bringing dreams across to reality.

Industrial issues

Specifiers tend to utilise only tried and tested materials and systems in construction for very human reasons. Despite many benefits from materials such as polymer composites, their exploitation to date has been limited to applications where their higher initial materials cost may be offset against unique benefits and the investment in empirical testing for future applications.

The extensive knowledge that we now possess about materials and systems has created new standards of admissibility for materials and systems. It allows us to begin to think more clearly about controlled innovation in the manner of all advanced industries.

At the moment in the UK there are major research initiatives to see how far construction can learn from manufacturing and other areas of advanced production engineering. One programme is by the Engineering and Physical Sciences Research Council as its Innovative Manufacturing Initiative. This has a specific programme for construction, called Construction as a Manufacturing Process. The Royal Academy of Engineering published the results of its study on the subject in 1995 in a report called "A Statement on the Construction Industry".

Remember that construction is not a backward form of manufacturing, which should look uncritically to say the car industry as a model for emulation. Consider some distinctive features of our sector:

- since construction is mostly fixed to the ground, it needs a mobile industry to reach each site and this imposes various hazards and uncertainties which make the process as unique as the delivered product.

- because they enclose activities constructions are bulky in their nature and are likely to remain so.

- in the building sector, we use thousands of different products and components, often only a few of each on any given project. This leads to an extraordinary range of production and assembly methods and consequent complexity of the process and of the product in contrast to the brilliant simplicity of many of the components. In civil engineering, we tend to use a smaller range of materials and components, but in larger quantities which present their own logistical problems. It often pays to study them in analytically more demanding ways

- since elements have to be long-lasting, durability is tricky as are the associated forms of finance and procurement. Society does not wish to wait a full life cycle before it claims the benefit of some new composite material.

- in a long-industrialised and urbanised country such as the UK, over half of construction is work to existing buildings/structures.

- construction is made from relatively cheap heavy materials compared to many other manufactured products. Construction materials cost about £0.2/kg and finished construction around £0.8/kg, compared with cars, where materials typically cost around £6/kg and the finished product around £12/kg. The methods of recovery, conversion, manufacturing, handling and transport are consequently distinctive.

  The comparison shifts when we consider the cost per cubic metre of created volume in the products of different industries, like houses, factories, reservoirs, cars, ships, trains or planes.

Construction is different and distinct from manufacturing, but we can still learn a lot from these other forms of advanced production, mostly in rethinking ***the whole project process*** and systematically incorporating controlled innovation.

We are now faced with a new synthesis of product and process with the developments in CAD-CAM, modelling and simulation of supply chain processes, the use of virtual reality and the use of single project databases as the unifying common information of the design team. These methods already exist in the aero-space, automotive and process plant industries. We should relish their impact upon our own sector. They will be crucial as we exploit the full potential of composite design in construction.

Simultaneously we see a renewed interest in the use of pre-assembly and standardisation, as we recognise their role in industrial change.

Composite design as a reconciliation of contradictions

The constituents of a composite solution must be compatible and the benefits of the composite approach should outweigh any disadvantages. Fortunately, steel and concrete have similar thermal expansion coefficients and for a period compatible surface chemistries. Polymer composite reinforcements and concrete move differently for thermal change. This extra effect must be overcome in effective design, a typical problem when mixing untried materials or systems.

An effective solution is not always achieved. Clinker as an aggregate for 19th Century concrete was cheap and readily available. It can however support combustion with difficulties for compartment floors and walls and create incompatibility problems when wet for embedded iron or steelwork, due to its content of sulphur.

For today's polymer composites, considerable unknowns remain regarding aspects like creep, fire performance and methods of joining to other components.

The construction industry must attempt to clarify such unknowns if only by appropriate safety factors to allow for problems in use. Use will then be more extensive and we will find worthwhile challenge in the unresolved contradictions.

The concept of controlled innovation has served in other industries. It is available for us in the construction sector to embrace more overtly. It would affect the industry in its organisation as well as in the components of construction.

The argument would deliver a virtuous circle driven by modern technological and industrial methods in which

- we increase our understanding of materials
- we develop our analytical techniques
- we develop our computational subtlety
- we redefine a larger and more complex proportion of the designed product as the analysable entity, inevitably more composite
- we add new materials to old constructions, to enhance their life and performance
- we understand and exploit the significance of greater precision of manufacture and new organisation of production
- we better define and then simulate both the product and the production processes
- we see prospects for changing the overall project process
- we can invent new composite structures and constructions
- we feel free to examine more materials, more performance attributes, more structural combinations

In this formulation, the concept of composite construction is the temporarily stable means by which we can get on with aspects of our work, including the resolution of inherent performance contradictions of materials or structural forms. The very stability sows the seeds of further change, through redefinition of the design task and eventually through its demands for ever more precise and controlled production.

The engine of the creative resolution of these current issues will be the new ideas from our designers.

Even with all these more powerful techniques, tools and understandings we have not eliminated the need for good engineers. It is like using a Stradivarius violin. If you give me one, I shall probably be concerned with not dropping it. If you give one to an orchestra musician, her performance will sound a bit better. But if you want to hear the difference it makes to use high class instruments, you need a world-class musician. So it is with engineering: to get the best out of the new tools, we need excellent engineering designers. Even then the designer is not to be set merely equal to the world class musician, whose art is a reproductive or at least an interpretive one. The designer has a clean sheet each time, and is limited only by his own mind and the other minds he can access.

I thank my colleagues at Arups for their help in developing this proposition.

High Strength Materials in Composite Construction

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Summary

High strength steels and high strength concretes have been used successfully in composite construction in Australia for some years. The use of high strength materials has developed through both practical considerations and careful research. High strength materials have been used in beams, columns and slabs to improve economy. It has been found that in addition to strength and serviceability, stability, local buckling and ductility are also important effects in the design of composite members incorporating high strength materials.

1. Buildings utilising High Strength Materials

1.1 Benefits

High strength materials offer new improvements in economy in composite construction. High strength materials also offer significant benefits in some specific areas of both concrete and steel construction. In composite construction similar benefits emerge, but the opportunity to combine materials creatively gives other opportunities and challenges.

Particular examples of the benefits in the use of high strength materials include:

- Use of high strength concrete in columns and core walls. Studies (eg. Sparrow [1], Rose & Martin [2]) have shown that increase in concrete strength has an economic benefit. Concrete up to 80 MPa (cylinder strength) is now commonly used in Australian building, while 100 MPa concrete can be supplied in some major cities. Using minimum reinforcement (1%) in conjunction with the highest strength concrete available consistently produces the most economic result. For core walls, this is further accentuated by the increase in rentable space produced by the reduction in structural sizes.

- Use of high strength steel in non-serviceability critical applications including radio masts.

Over the past seven or eight years, Connell Wagner has carried out structural designs for numerous major multi-storey buildings around Australia. Many of these are in the 40 storey plus category. In addition, concept and/or advanced designs have been carried out on very tall buildings such as the 72 and 84 storey schemes for Melbourne Central and options for three supertall buildings in Brisbane. Some advantages of the use of high strength concrete in high rise buildings are given in Table 1.

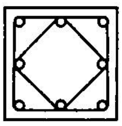
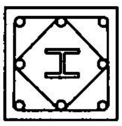
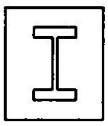
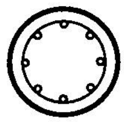
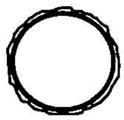
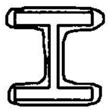
In steel structures, the grades of steel most commonly available have now been raised from 250 MPa and 350 MPa to 300 MPa and 400 MPa respectively providing economies whenever serviceability requirements are not critical. This is also reflected in composite beams for which deflection is not generally critical, provided construction deflection of unpropped beams is controlled by precambering the steel section.

Requirement	Solution
Reduced member sizes, foundation loads	Smaller lighter members
Increased rentable floor space	Reduced core wall thickness and column dimensions
Reduced cost	Less material and easier handling
Wind sway control	Increased column flexural stiffness
Reduced differential shortening	Potential for reduced shrinkage and creep
Early stripping	Strength achieved in shorter time

Table 1 Advantages of high strength concrete in high-rise building construction

1.2 Economics

One of the more interesting aspects of composite construction is in the design of composite columns. Connell Wagner performed many comparisons of column economy during the late 80's and early 90's. These were times of high timber formwork costs when steel systems are particularly attractive. These studies showed the steel tube filled with concrete to be an economical construction medium, which matched the concrete column, and gave other advantages. These columns were used very successfully on the Casselden Place project [3] and several subsequent projects in the early 90's.

No of storeys	**Type 1** Reinforced concrete	**Type 2** Reinforced concrete, steel erection column	**Type 3** Concrete encased steel section	**Type 4** Exposed steel tube filled with reinforced concrete	**Type 5** Fire-sprayed steel tube filled with concrete	**Type 6** Fire-sprayed steel section
						
10 levels	450x450 8Y32	450x450 8Y20 200UC46 Grade 350	410x410 310UC118 Grade 350	500 dia 6Y20 500x6.4 Grade 250	500 dia 500x6.4 Grade 250	310UC240 Grade 350
Relative Cost	1.0	1.22	1.53	1.14	1.10	2.27
30 levels	750x750 20Y36	750x650 12Y36 250UC89 Grade 350	570x570 400x50 flange 360x25 web Grade 350	800 dia. 6Y32 800x10 Grade 250	800 dia. 800x10 Grade 250	Plate girder 500x60 flange 460x40 web Grade 350
Relative Cost	1.0	1.13	1.85	1.11	1.02	2.61

Note: Loaded by 8.4m x 8.4m bays of steel framing

Table 2 Comparison of different column construction options

Costings for a series of alternative column configurations to resist the same load are given in Table 2 (Webb and Peyton [4]). While these costings are not necessarily up to date, they do illustrate some important points eg. a steel circular tube column filled with high strength concrete can be cost effective compared to the all steel column.

Another interesting feature is revealed in Figures 1 and 2. For a constant load carrying capacity for a given concrete strength, the cost of reducing the overall size of a reinforced concrete column using additional reinforcement is very high and becomes prohibitive for higher strength concrete (Figure 1). This phenomenon is not nearly so marked in a tube column (60 MPa concrete) where similar size reductions using increased plate thickness can be achieved with only modest cost penalty (Figure 2).

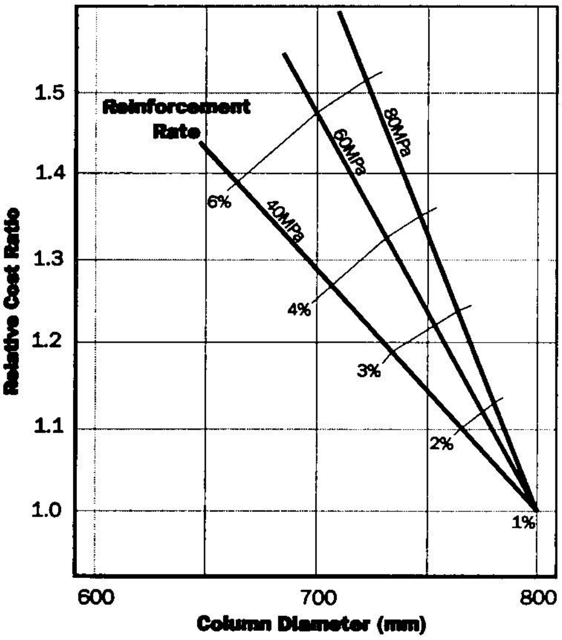


Figure 1 Relative costs for reinforced concrete columns with constant load capacity and varying percentage reinforcement

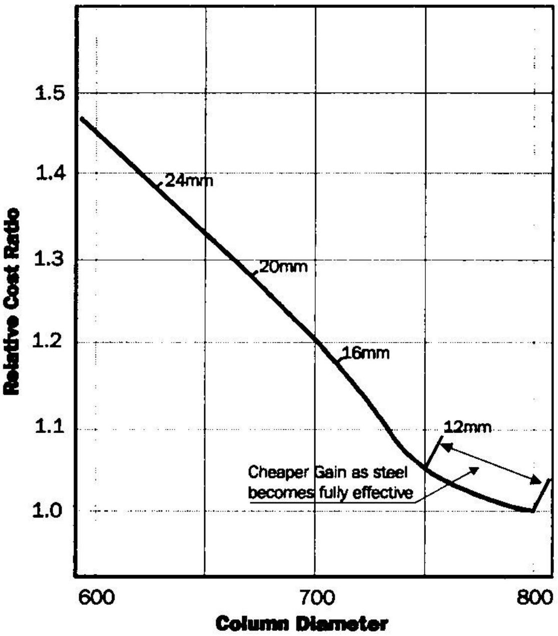


Figure 2 Relative costs for concrete-filled tubes with constant load capacity and varying wall thickness

Tube columns also provide a better environment for high-strength concrete because:

- adequate compaction is possible using pumping techniques alone,
- good curing conditions exist inside the tube,
- minimal creep and shrinkage occur in the effectively sealed conditions with minimal moisture loss, and
- the ductility of the concrete is improved, particularly for the thicker tube walls.

Eurocode 4 [5] also recognises the improved conditions of concrete in a filled tubular section by removing the 0.85 factor on the concrete strength.

1.3 Constructability

The concrete-filled steel tubular column is seen as an attractive viable system because it provides the ability to construct a composite building with the benefits normally associated with traditional steel construction. It is a natural efficient method for using high strength concrete. Research [6] is now underway using these columns with 100 to 120 MPa concrete as outlined in Section 2. The economy of this construction technique is driven by using the minimum amount of reinforcement and steel in the tubes with the highest strength concrete.

Placing concrete for these columns using a pumping technique has been pioneered on the Casselden Place project. The concrete is pumped into the steel tubes through a nozzle connection. In any one lift, concrete is placed from the bottom and pumped up as many as six stories at a time. The placement method, which eliminates the need to vibrate the concrete, was validated by a full-scale prototype test [3].

The tube is erected much like a traditional steel column, with floor reinforcing, concreting, etc. occurring as in conventional steel building practice. The tubes are erected in either two-or three-storey lengths and connected temporarily by turnbuckles. These facilitate plumbing and alignment and allow the crane to quickly release the column. This technique can also achieve excellent tolerances in column plumbing and alignment. The splice is completed using butt-welding, and the turnbuckles are then removed.

Concrete with strength up to 70 MPa (10,000 psi) was used on the project. The superplasticised concrete mix contains silica fume, principally to eliminate bleeding, thereby providing consistency over the height of the placement and eliminating the need to scabble the interface between placements. The method is extremely efficient, and large numbers of columns can be filled at a time by a small workforce and with minimal material waste.

The bare steel tube was capable of supporting six floors of construction, giving the contractor flexibility as to when and where to place the concrete.

2. Steel Tubes filled with High Strength Concrete

2.1 Concrete Stress-Strain Characteristic

High strength concrete is particularly economical when used in circular concrete-filled tubes subjected mainly to axial compression for which the high compressive strength can be fully utilised in design. To obtain this economy, it is essential that the highest strength can be achieved still using conventional materials and manufacturing and placement procedures.

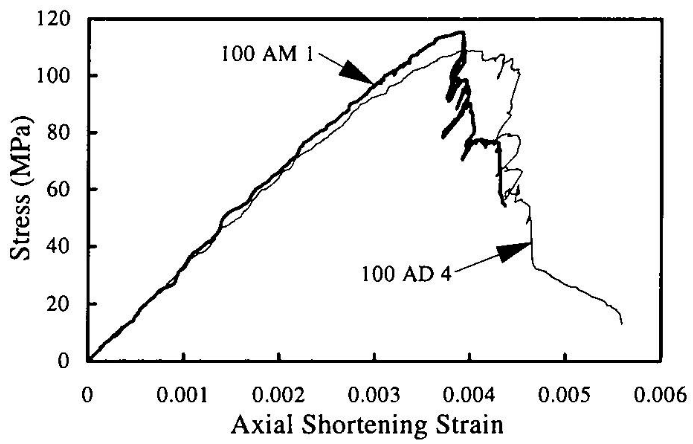


Figure 3 Stress-strain curve for high strength concrete from cylinder tests.

A commercial concrete mix has been developed that meets these objectives and can develop a compressive strength around 120 MPa. However, the post-ultimate stress-strain response of high strength concrete can be characterised by a very rapid unloading, even exhibiting the phenomena of "snap-back". This brittle behaviour cannot be measured using normal testing procedures including displacement control. A number of alternative techniques can be used. A simple method is to use the diametric strain to control the displacement of the testing machine as this was found to increase monotonically with axial displacement. The results of such a test are shown in Figure 3. The concrete cylinder labelled with an "M" was moist-cured in a lime bath at

100% relative humidity and a constant temperature of 20^{0}C. The cylinder labelled with a "D" was dry-cured by sealing the cylinder in a polythene wrap and storing it at ambient temperatures to simulate conditions in the concrete-filled tube specimens.

2.2 Concrete-filled Steel Tubes - Steel and Concrete Loaded

For concrete-filled steel tubes in which the concrete and steel are loaded simultaneously, enhancement of concrete strength due to confinement by the tube can be obtained for low to medium strength concretes and this has been recognised in Eurocode 4 [5]. The cross-sectional strength N_{u0} in axial compression including the beneficial effects of concrete confinement for concretes with strengths up to a maximum of 50 MPa is given by

$$N_{u0} = A_s \eta_2 f_y / \gamma_s + A_c f_c / \gamma_c \left(1 + \eta_1 (t/D)(f_y / f_c)\right) \tag{1}$$

where A_s and A_c are the cross-sectional area of the steel and the concrete, f_y and f_c are the characteristic strengths of the steel and concrete, and γ_c and γ_s are the partial safety factors which may be taken to be unity when the strength of the materials has been accurately measured as in laboratory tests. The increase in concrete strength from confinement (accounted for by the η_1 factor) and the corresponding decrease in steel strength (accounted by the η_2 factor) may be considered if the non-dimensional column slenderness λ is less than 0.5 and the eccentricity of loading does not exceed $D/10$ where D is the external tube diameter.

Recent axial load tests by O'Shea and Bridge [7] on steel tubes filled with high strength concretes with strengths in excess of 100 MPa have revealed that virtually no enhancement can be obtained, the concrete behaving as if unconfined up to the maximum strength. Five tubes designated CS were tested under axial load. Their dimensions and material properties are given in Table 3 together with the maximum load capacity N_{Test} obtained in the test.

Tube	Diameter (mm)	Thickness (mm)	Length (mm)	Average Concrete Strength (MPa)	Steel Yield Stress (MPa)	Max. Load N_{Test} (kN)
S30CS	165	3.00	578	113.5	364	2673
S20CS	190	2.00	660	113.5	272	3360
S16CS	190	1.55	662	113.5	315	3260
S12CS	190	1.15	660	113.5	185	3058
S10CS	190	0.95	662	113.5	211	3070
S20CL	190	2.00	654	113.5	272	3690
S12CL	190	1.15	662	113.5	185	3220

Table 3. Specimen dimensions and properties

In Table 4, code predicted strengths N_{Code} (Equation 1) are compared to the actual test strengths N_{Test}. The code values for η_1 and η_2 are calculated using Eurocode 4 [5]. Using these values, the equivalent factor implied in the code for concrete strength enhancement (a factor of 1.0 being zero enhancement) can be calculated and is shown in Table 4 under "Concrete Enhancement EC4", the thicker tubes having more enhancement. The Grimault and Janss [8] effective steel area was used in the calculations.

An approximate value of the concrete enhancement factor in the tests can be calculated by subtracting the maximum bare steel strength from the maximum concrete-filled steel tube capacity and dividing the remainder by the concrete area and the concrete cylinder strength. This is shown in Table 4 under "Concrete Enhancement Test". The values shown are all slightly less than unity suggesting that there is little concrete enhancement (or alternatively the concrete cylinder strengths are inaccurate). As the actual concrete strength was determined from at least ten cylinder tests, it is more likely that there is little confinement of the concrete in the steel tubes. This is also supported by the fact that the test values are essentially constant and do not increase with tube wall thickness. If no confinement is assumed i.e. letting $\eta_2 = 1.0$ and $\eta_l = 0$, Eurocode 4 [5] provides a good prediction of the section strength as shown in the last column of Table 4. When confinement is considered, Eurocode 4 [5] is slightly unconservative for axially loaded thin-walled steel tubes infilled with high strength concrete. Therefore, confinement for high strength concretes with strengths in the range 100 MPa and above should be ignored.

Specimen	η_2 steel EC4	η_l concrete EC4	Concrete Enhancement EC4	Concrete Enhancement Test	N_{Test}/N_{Code} Confinement EC4	N_{Test}/N_{Code} No Confinement EC4
S30CS	0.866	1.514	1.083	0.949	0.927	0.966
S20CS	0.872	1.390	1.033	0.996	0.982	1.000
S16CS	0.879	1.265	1.025	0.969	0.967	0.980
S12CS	0.883	1.183	1.011	0.939	0.946	0.952
S10CS	0.888	1.084	1.008	0.942	0.957	0.961

Table 4. Comparison of test results with Eurocode 4 [5]

2.3 Concrete-filled Steel Tubes - Concrete Loaded

Two circular tubes designated CL were tested [7], S20CL and S12CL, with only the concrete loaded. Their dimensions are given in Table 3 and were similar to their companion S20CS and S12CS tubes. The concrete was loaded axially with the steel unbonded. This was achieved through greasing the internal tube surface prior to filling with concrete. Strains measured on the steel tubes using rosettes verified the debonding procedure. Special loading disks were manufactured to ensure that the axial load was only applied to the concrete. Therefore the steel only provided lateral confinement to the concrete and did not carry any direct axial load.

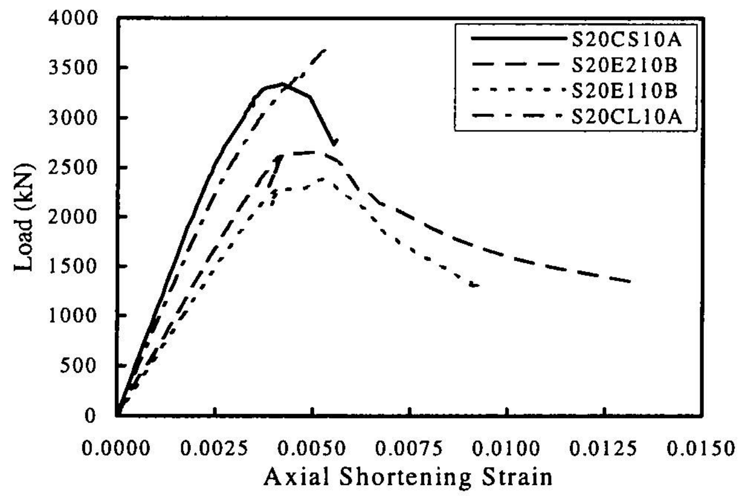


Figure 4 Load - axial strain response for S20 concrete-filled steel tubes

The load-axial strain response for the axially loaded S20CL tube is compared with that for the companion S20CS tube in Figure 4. Also shown is the response for companion eccentrically loaded tubes S20E1 and S20E2 with eccentricities of D/10 and D/20 respectively. These have a more ductile response than the axially loaded tubes.

It can be seen that the maximum load for the concrete loaded only specimen S20CL was higher than that for specimen S20CS for which the concrete and steel were loaded simultaneously. The same behaviour was observed for the S12 tubes as indicated in the last column of Table 3. Therefore, an increased strength can be obtained without the steel being axially loaded. The strength increase was 9.8% for the thicker S20 tubes and 5.3% for the thinner S12 tubes, the thicker tube providing more confinement and hence higher strength increase as expected. Hence, confinement of high strength concrete is possible provided the concrete alone is loaded.

The more efficient use of steel tubes filled with unbonded concrete with only the concrete loaded has been proposed by Orito et. al. [9]. However, detailing, especially at beam to column joints, needs careful consideration to ensure axial load is not transferred to the steel tube.

2.4 High Strength Steels - Local Buckling

With the use of high strength steels, thinner steel sections can be achieved for the same load carrying capacity. However, the effects of local buckling have to be considered. O'Shea and Bridge [7] have examined the local buckling of circular steel tubes with or without concrete infill. The results of axial load tests on bare steel tubes (BS tests) and steel tubes with unbonded concrete infill with only the steel loaded (BSC tests) are shown in Table 5.

Specimen	Test bare steel (kN)	Test concrete infill (kN)	AS4100 [10] (kN)	Grimault & Janss [8] (kN)
S30BS/C	523.3	521.6	510.4	521.6
S20BS/C	284.5	279.9	270.9	304.3
S16BS/C	239.2	283.8	202.0	252.1
S12BS/C	109.1	109.3	100.5	118.2
S10BS/C	92.9	91.0	71.4	91.2

Table 5 Capacity of steel loaded circular tubes

It was found, in general, that the concrete infill did not enhance the local buckling strength as the local buckle was a circumferential "elephant's foot" buckle occurring at one end of the tube. This outwards buckle is not restrained by the concrete infill which therefore has no effect. As shown in Table 5, the local buckling strength for bare steel tubes can also be predicted reasonably accurately using the design rules in current steel codes [10] and the literature [8].

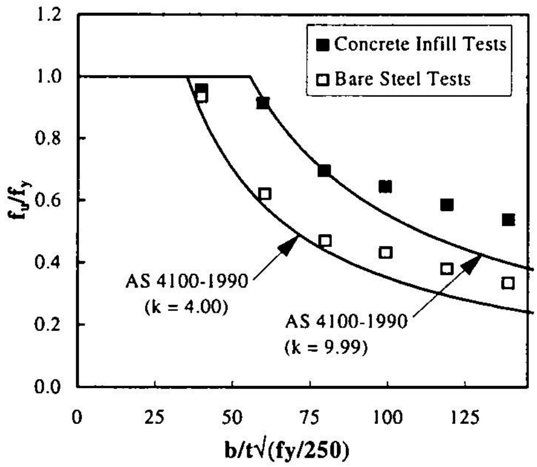


Figure 5 Strength of steel-loaded square tubes

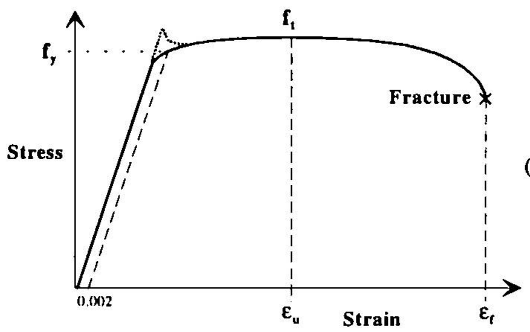
Bridge and O'Shea [11] have also examined the effects of local buckling for square thin-walled steel tubes with or without concrete infill. For the bare steel tubes, the local buckling pattern exhibits both inwards and outwards buckling deformations and the plate elements forming the walls can be considered as having simply supported edges with a buckling coefficient *k* of 4.0. The concrete infill prevents any inwards buckling and the plate elements can be considered as having clamped edges with a buckling coefficient k of 9.99. This results in an increase in column strength as evidenced in the test results plotted in Figure 5 where b is the width of the tube and t is the plate thickness. Also, current design methods in standards such as AS4100-1990 [10] can be used to predict the plate strength f_u of such tubes provided the correct value of buckling coefficient k can be included in the design method.

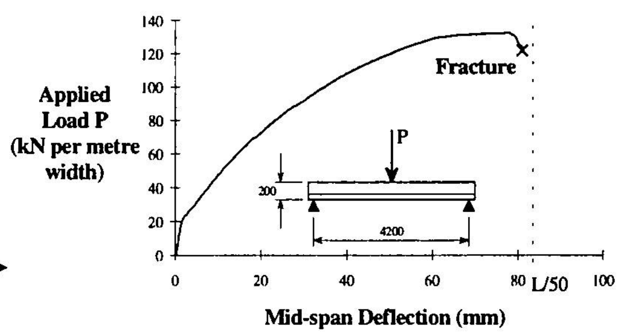
3. Composite Slabs utilising Galvanised High Strength Sheet Steels

Australian profiled steel sheeting has been consistently manufactured from galvanised, high-strength (f_y = 550 MPa) sheet steels for over 30 years. Research by Australian industry has ensured that the major Australian profiles develop strong mechanical resistance with the concrete. In conventional composite slabs in steel-frame buildings, which typically involve unpropped spans of about three metres and depths of less than 200 mm, the thickness of the sheeting is often determined at the formwork stage when the high yield stress of the steel can be utilised. This can result in the tensile capacity of the sheeting being significantly under-utilised in the design of the composite slab. However, G550 steel can be used most effectively as the overall depth, span and applied load increase. Long-spanning composite slabs in bandbeam construction may have depths up to 400 mm.

3.1 G550 Sheet Steels

The G550 sheet steels used to make profiled steel sheeting range in thickness from 0.6 to 1.0 mm with a total zinc coating mass of 200 to 450 g/m^2. Steels of such high strength are not normally used for this purpose in Europe or America where hot-rolled grades of between about 275 and 350 MPa predominate. Some potentially undesirable properties of G550 steels are their low elongation at fracture and lack of strain-hardening.





(a) Stress-strain curve from coupon testing *(b) Composite slab load-deflection curve*

Figure 6 Tensile testing of G550 sheet steels

3.1.1 Material Standards

Cold reduction is used rather than an alloying process to produce the G550 sheet steels from hot-rolled 2.5 mm thick steel which has a minimum specified yield stress of 300 MPa. The material property requirements for G550 sheet steels are specified in AS1397 [12]. The minimum yield stress and tensile strength are both 550 MPa, and the minimum elongation in either a 50 mm or 80 mm gauge length is 2 per cent.

3.1.2 Stress-Strain Curve

Coupons taken in the longitudinal direction of a coil and tested at a low strain-rate exhibit a stress-strain curve of the form shown in Figure 6(a). Material properties are calculated assuming only the base metal is present. The G550 sheet steels may exhibit an upper yield (dashed peak in Figure 6(a)) or else yield gradually (depending on processing after coating), in either case with minimal strain hardening. Yield stress f_y is calculated using the 0.2% strain offset method, and values of the tensile-strength-to-yield-stress ratio (f_t/f_y) equal to unity are consistently obtained. The modulus of elasticity is typically about 200 GPa, and this value can be used for design.

Yield stress values for G550 sheet steels are significantly above the minimum specified 550 MPa. Mean strengths tend to increase as the base metal thickness reduces due to the increased amount of cold reduction. Large variation occurs in the strength and ductility of material taken from different mills. Post-ultimate ductility measured by the fracture-to-ultimate-load ratio is comparable to that for mild sheet steels and can reach as low as 0.7.

3.2 Australian Profiled Steel Sheeting

The major Australian profiles have two important similarities, viz.: (1) the steel ribs are very narrow in width compared with the flat pans, and therefore a transverse section of a slab closely resembles that of a solid slab; and (2) the lapped ribs formed when sheets are joined together on site are specially shaped to develop strong, ductile mechanical resistance with hardened concrete.

3.2.1 Mechanical Resistance developed with Concrete
The Slip-Block™ Test has been specially developed in Australia to measure mechanical resistance τ as a function of slip after the breakdown of adhesion bond [13]. The coefficient of friction μ developed between the sliding surfaces is also measured during the test. Both these parameters are used in partial shear connection theory for designing composite slabs [14].

Tests on the Australian profile Bondek® II have yielded design values $\tau = 115\sqrt{(t_{bm}f'_c)}$ in kPa and $\mu = 0.6$, where t_{bm} is the nominal base metal thickness of the sheeting (mm) and f'_c is the characteristic compressive strength of concrete at 28 days (MPa) [15]. The tops of the sheeting ribs are embossed which contributes significantly to the magnitude and ductility of the mechanical resistance they develop. Embossing must not cause any local fracturing in the G550 sheet steel, since this can lead to premature fracture of sheets when they act as reinforcement at the composite stage.

3.2.2 Quality Control
Profile features critical to the development of mechanical resistance (e.g. lap joint shape, embossments, etc.) must have their geometries adequately maintained by an operational quality-control program. Additional variations that can result because the profile is manufactured at different sites must be considered when establishing minimum production standards.

3.2.3 Residual Stresses
When a profile is roll-formed from coils of G550 sheet steel, large residual stresses typically of ±100 MPa are induced in the sheeting which vary across its width.

3.3 Design of Conventional Composite Slabs

The slabs may be designed as simply-supported for some limit states and continuous for others, subject to certain conditions being satisfied. Reinforcing steel is placed transversely in the slabs for shrinkage and temperature control. Longitudinal reinforcement may be placed in the top face over supports to increase negative moment capacity and control flexural cracking. It may also be placed in the bottom face, where (because the Australian profiles develop strong, ductile mechanical resistance) it can act in conjunction with the sheeting and increase the positive moment capacity. A set of limit-state design rules has recently been prepared to Australian Standards for composite slabs incorporating Bondek II [15] and are briefly discussed below.

3.3.1 Design for Strength
Design for strength is concerned with ensuring that there is sufficient moment and shear capacity in both positive and negative moment regions. Elastic analysis may be used to calculate the design action effects of continuous slabs, and the amount of redistribution permitted depends on the ductility of hinge regions and is influenced by both the amount of the steel reinforcement and its tensile properties. If high-strength, low-elongation reinforcement is used over supports, either no redistribution is permitted [16], or the slabs must be assumed to revert to simple spans at

ultimate load due to possible fracture of the reinforcement (100 per cent redistribution). Premature fracture of low-elongation reinforcement in the bottom-face acting in conjunction with the steel sheeting may also be a problem and may need to be considered in design.

Partial shear connection strength theory can be used to design the positive moment regions of composite slabs exhibiting one-way action. Simplified equations have been formulated to calculate the design positive moment capacity of Bondek II slab cross-sections with either partial or complete shear connection and including the contribution of bottom-face reinforcement. The model used in the theory recognises the existence of support friction and the additional anchorage of the sheeting continuing into adjacent spans. The accuracy of the model has been validated by testing [17, 18].

In one such test, the theory was used to predict the failure load of a Bondek II slab (see Figure 6(b)). The test showed that the full moment capacity of the peak moment cross-section (corresponding to yielding of the steel sheet over its full cross-sectional area) was reached. The stress-strain curve of the steel (t_{bm} = 0.750 mm) in the longitudinal direction took the form shown in Figure 6(a) with an upper yield point. Tensile coupons from the steel gave varied results with yield stress f_y ranging from 630 to 635 MPa, upper yield stress from 640 to 660 MPa, and elongation after fracture (50 mm gauge length) from 5.6 to 8.5 per cent. Interestingly, however, uniform elongation (37.5 mm gauge length outside fracture zone) was close to zero for coupons with a high upper-yield-stress-to-yield-stress ratio, while it reached a maximum of nearly 5 per cent for other coupons. In the slab test, failure occurred when the steel fractured at a mid-span deflection of almost span/50, which is acceptable considering that a single line-load was used in the test and there was a steep moment gradient. Fracture occurred at a major flexural crack and extended across a whole sheeting pan and through an adjacent rib. The peak of the curve in Figure 6(b) could be predicted very accurately. More complex situations involving the effects of partial shear connection and support friction have also been accurately predicted (within 10 per cent) using the theory [18]. The residual stresses described in Section 3.2.3 created during roll-forming can be ignored in the analysis, as can any initial flexural stresses arising during construction prior to composite action.

The shear capacity of the positive moment regions of Bondek II slabs has been investigated experimentally [19], and it has been recommended that for this profile the occurrence of vertical shear failure (viz. by diagonal splitting or flexure shear) can be ignored under uniform-loading conditions. This recommendation applies only when the amount of reinforcing steel taken into account in determining the moment capacity of the slab is less than a certain amount [15].

3.3.2 Design for Deflection Control

For normal situations experienced in steel-frame buildings, design is satisfactory treating the sheeting as fully effective reinforcement and using methods applicable to reinforced-concrete for calculating section properties. Limits on total and incremental deflections (Δ_{tot} and Δ_{inc}) are normally specified. However, design for deflection control of Bondek II composite slabs in long-spanning applications is currently under review and a variety of tests are being performed.

3.3.3 Design for Crack Control and Durability

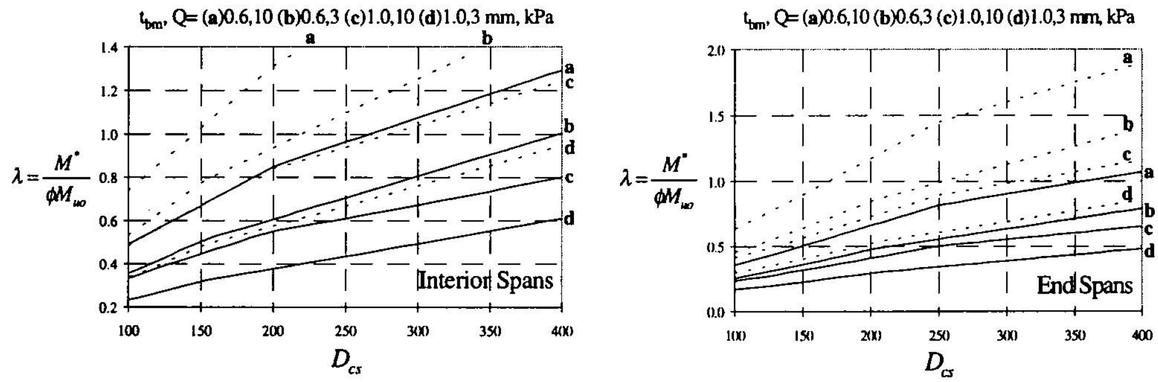
The negative moment regions of a composite slab are susceptible to top-surface flexural cracking. Controlling such cracking is particularly important as excessive crack widths can give an overall impression of poor quality and can limit the types of floor coverings that can be successfully used. Excessively wide cracks can also provide a pathway for the ingress of corrosive substances such as water. As a general rule, a designer should aim to detail the member such that under service loading the tensile strain at the top surface will be distributed over a large number of narrow cracks rather than a small number of wide cracks.

The main design principles are: (i) crack width is calculated using a simplified equation derived using the method for calculating crack width defined in BS 8110, Part 2 [20]; (ii) a limiting width of 0.3 mm specified in BS 8110, Part 2 is adopted; (iii) elastic analysis is used to calculate design bending moments under both short-term and long-term service loading; (iv) the stress in the reinforcement is calculated using elastic, cracked-concrete modular ratio theory and is kept below the yield stress under short-term service loading and 80 per cent of the yield stress under long-term service loading; and (v) the nominal negative moment capacity of the support regions is to exceed the cracking moment by at least 20 per cent so that more than one flexural crack will form.

3.4 Factors affecting Utilisation of Sheet Steel Tensile Capacity

Bandbeams are wide, shallow beams that are supported in a parallel arrangement on isolated columns. Slabs are formed between the bandbeams. Both the slabs and bandbeams are assumed to exhibit one-way action. While slab depths in steel-frame buildings seldom exceed 200 mm, slabs spanning between bandbeams can reach 400 mm or more. The span-to-overall-depth ratio of a slab (L/D_{cs}) in either of these situations typically varies between 20 and 40, depending on the support conditions, magnitude of loading and deflection limits.

The design rules described in Section 3.3 have been used to prepare sets of solutions for examining the effect of base metal thickness, live load, yield stress, moment redistribution and slab support conditions on the utilisation of sheet steel tensile capacity in Bondek II slabs. For brevity, only some of the results are presented here. Slab overall depth was considered to vary from 100 to 400 mm. All solutions satisfy the criteria concerned with design for strength, deflection control ($\Delta_{tot}/L \leq 1/250$ and $\Delta_{inc}/L \leq 1/350$), and crack control when appropriate. Utilisation is examined for the positive moment region using the ratio of design positive bending moment to design positive moment capacity, i.e. $\lambda = M^*/(\phi M_{uo})$. The sheeting is assumed to be the only bottom-face reinforcement, and it is assumed to be fully anchored at the critical cross-section. Cases for which $\lambda > 1$ imply that the tensile capacity of the sheeting is fully utilised and additional bottom-face reinforcement is required for strength.



(a) Yield stress varied from 550 to 350 MPa *(b) Moment redistribution of 100%*

Figure 7 Factors affecting utilisation of G550 sheet steels in Bondek II slabs

The cases shown in Figure 7(a) are for interior spans. The solid and dashed lines are for $f_y = 550$ and 350 MPa, respectively. Solutions for different combinations of base metal thickness t_{bm} and live load Q are also labelled. It can be seen that G550 steel is most effectively utilised in situations corresponding to bandbeam construction when depths are large (>250 mm), live load is heavy (10 kPa) and base metal thickness is least (0.6 mm). However, the solutions for G350 steel show that its tensile capacity can be exceeded in steel-frame building situations and hence bottom-face reinforcement would be required to supplement the sheeting.

The cases shown in Figure 7(b) are for end spans using only G550 steel The solid and dashed lines are for 0% and 100% redistribution respectively from negative to positive moment. The values of λ can be seen to increase substantially with moment redistribution indicating better utilisation of the high strength G550 steel. (Crack control reinforcement must be provided separately for this case if required.)

The effects of partial shear connection on the curves in Figure 7 can be ignored for slabs incorporating Bondek II on account of the strong mechanical resistance it develops.

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Outstanding Composite Structures for Buildings

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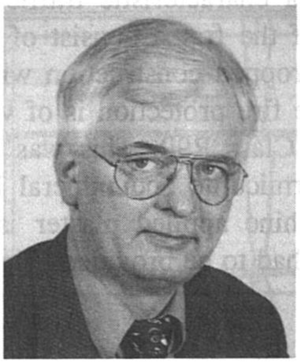

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Summary

The paper gives an overview of the development of composite structures for buildings in Germany in recent years. The background and advantage of partially concrete encased columns and girders is demonstrated with three buildings for the car industry, an office building and the extension of the airport in Hannover. Furthermore two impressive high rise buildings in Düsseldorf and Frankfurt will be presented.

1 Introduction

Comparing the development of composite structures for buildings in Europe it is obvious that in contrast with neighbouring countries, in Germany the technology with partially concrete encased members has been favoured over the last twenty years. The paper will show the advantages of this technology for some typical examples of outstanding composite structures for industrial and office buildings.

2 Buildings for the Car Industry

2.1 General

Industrial buildings for the car industry require a high flexibility because the technical equipment must be converted frequently during the design life. The Figures 1 to 3 show three typical buildings for the car industry built between 1982 and 1992. These buildings demonstrate the development of composite structures from the conventional type to the modern technology of concrete encased beams and columns with significant advantages, regarding fire resistance as well as durability and last but not least economic benefit.

2.2 Paint Unit of Opel in Rüsselsheim

In 1981 the Opel corp. erected a new paint unit in Rüsselsheim, shown in Figure 1 /1/. The building measures 405 x 80 metres and has three stories with a total height of 31.5 metres. Additionally a penthouse with dimensions of 340 x 20 metres with a height of 7 m is located between gridlines D and E. In the transverse direction the structure consists of sway frames with rigid connections at the top of each storey level on gridline C and with all the other connections nominally pinned. In the longitudinal direction the structure is stabilised by vertical bracings. For the composite slabs with a depth of 200 mm Holorib sheeting is used. The slabs with a span of 3.33 metres are supported by composite beams. Both slabs and composite beams had to be designed for a characteristic traffic load of 12,5 kN/m^2. The composite girders and the steel columns of the frame consist of welded sections The span length of the composite slabs required a propped construction with scaffolding truss girders according to Fig. 1. For a painting unit the fire protection is of vital importance. The whole structure is protected for a fire resistance Class R90. This was achieved by fire resistance boards and machine applied plaster on vermiculite and mineral basis. For the fastening of installations and suspended loads the machine applied plaster is especially unfavourabely. Therefore additional suspended steel beams had to be provided (Fig. 1).

2.3 Body Unit of Porsche in Stuttgart

Only 4 years later the new body unit of Porsche was built in Stuttgart (Fig. 2). The design philosophy had completely changed. In the meantime a code for composite columns had been published in Germany and intensive research work on fire resistance of partially concrete encased beams had been carried out. The building with three stories and an additional penthouse measures 125.5 x 65 m. The structure is stabilised in the transverse direction by concrete gable walls and in the longitudinal direction by truss bracings. Main and secondary beams are continuous and concrete partially encased sections are used; the columns are nominally pinned at both ends and fully encased composite sections are now used. The building has an extremely high degree of installation, requiring a lot of web openings in the main girders with a span of 20 metres. The highly stressed main girders have flanges and webs with a wide variation of plate thickness (Fig. 4). In comparison to the Opel paint unit the spacing of the secondary beams is reduced to 2.5 m to avoid propped construction for the composite slabs with a depth of 240 mm. The structural fire design was based on a model where the effect of temperature on material characteristics is taken into account by reducing the material properties and the dimensions of the cross sections. For the required fire resistance Class R90, additional reinforcement was necessary in the concrete encasement of the main and secondary beams. This reinforcement was not taken into account for normal temperature design.

2.4 Paint Unit of Opel in Eisenach

In 1992 the new Opel paint unit in Eisenach was one of the first major investments in the new states of reunified Germany (Fig. 3). The building is one of the most modern of this type in Europe and sets new milestones for the car industry /3/. The structure with measurements of 240 x 56 metres and up to 30 m high consists of two main production levels and an additional technical penthouse. The experiences with the body unit in Stuttgart showed that systems with nominally pinned columns lead to disadvantages during erection, because an enormous number of temporary bracings is necessary. Therefore a structural system with sway composite frames in the transverse direction and truss bracings in the longitudinal direction was chosen in Eisenach.

In order to minimise the time of construction and welding on site, the detailing of the connection is very important. Figure 3 shows the main frame joint used on gridlines B and C and the support of the main girders at the corner columns on gridlines A and D.

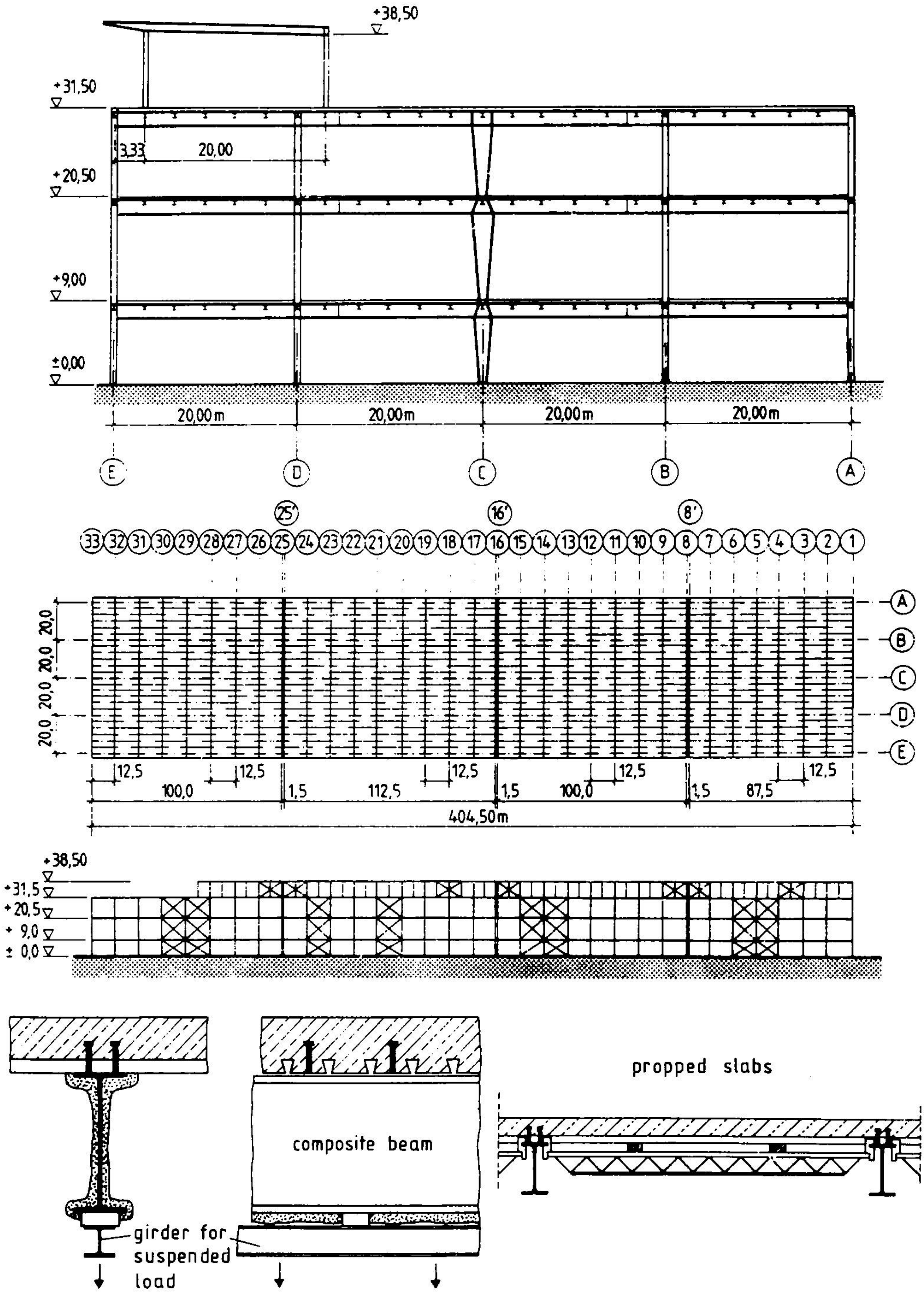


Fig. 1 Paint unit of Opel in Rüsselsheim (1981)

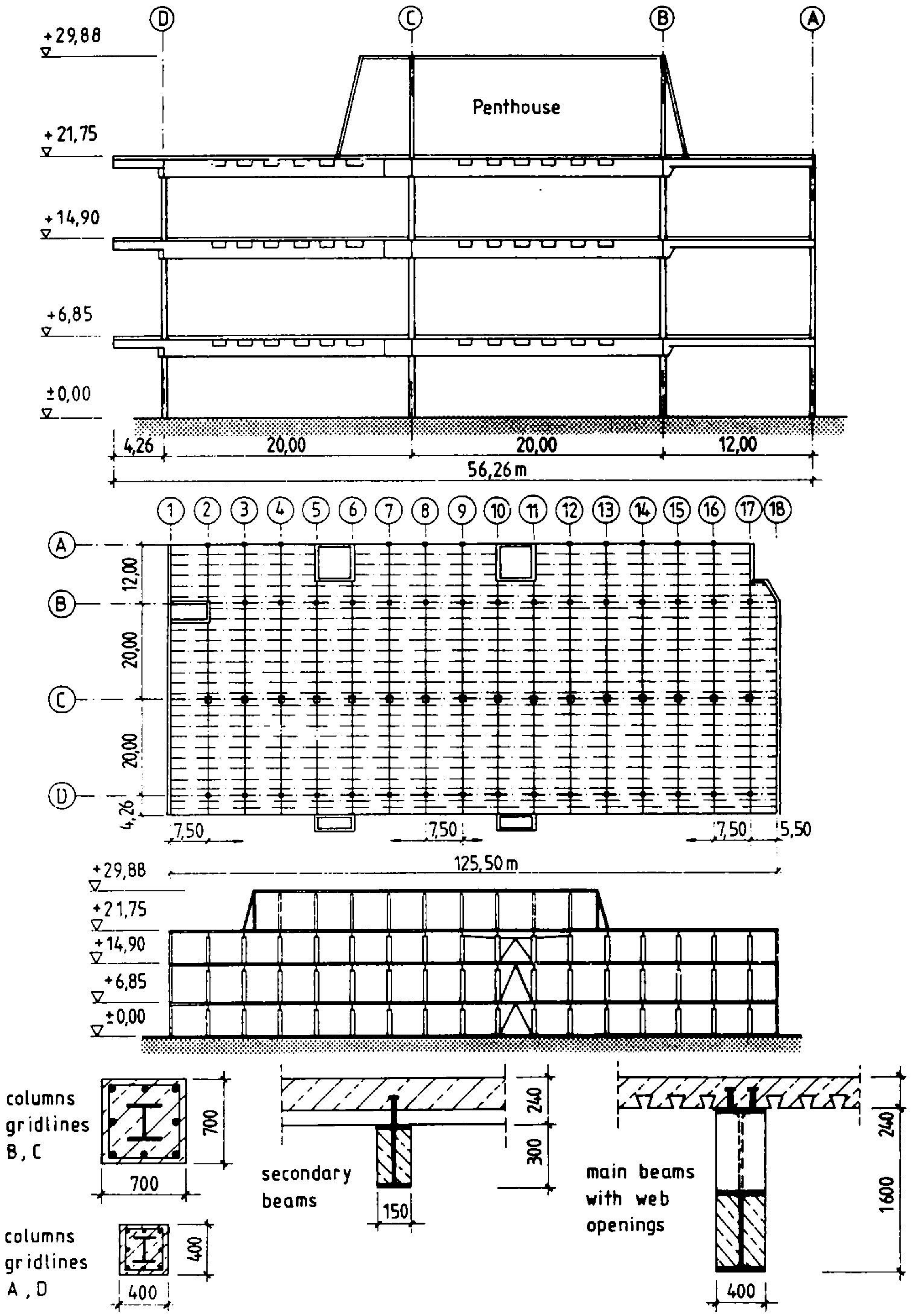


Fig. 2 Body unit of Porsche in Stuttgart (1985)

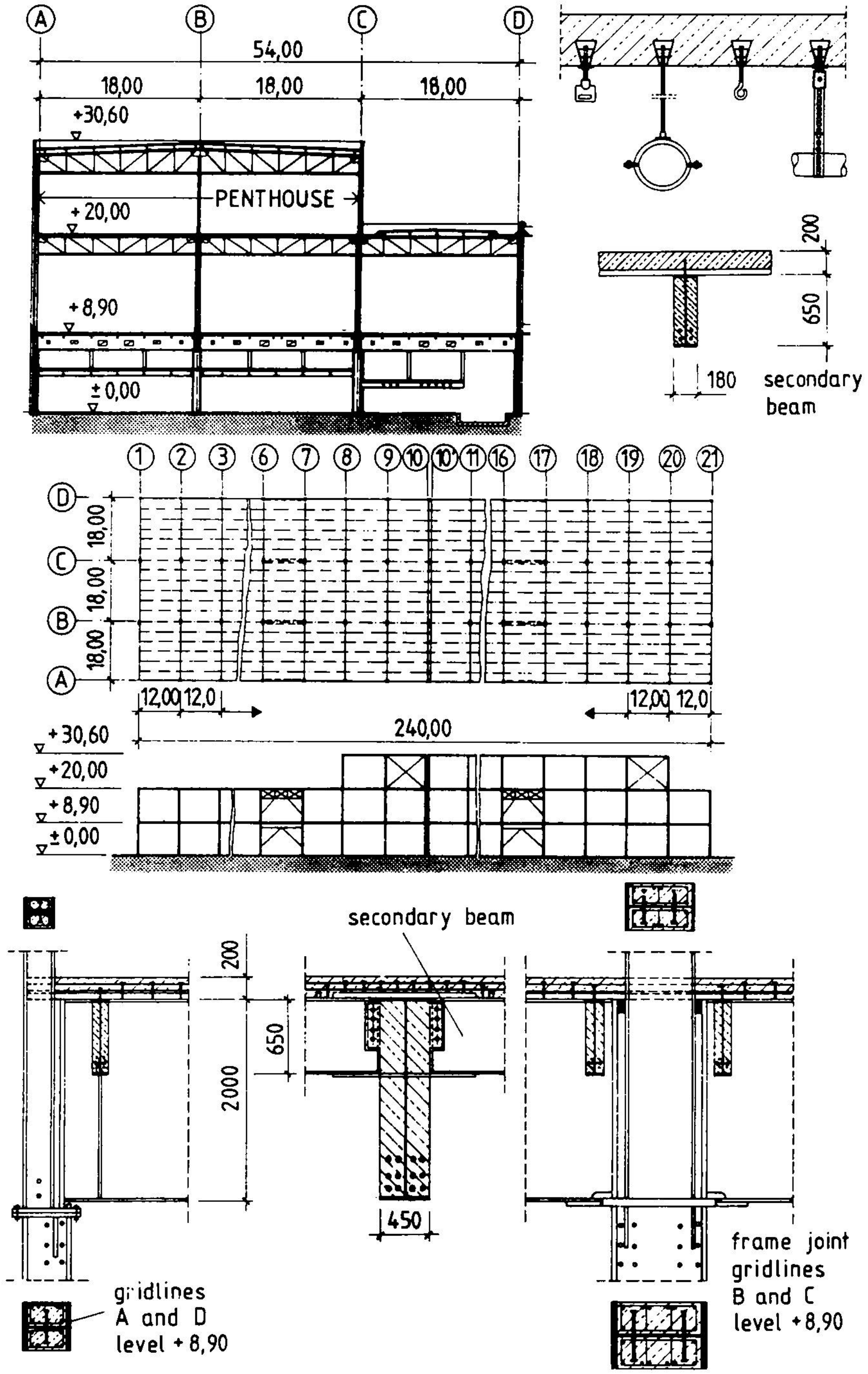


Fig. 3 Paint unit of Opel in Eisenach (1992)

The depth of the columns changes at each storey level, and the supporting reactions of the main girders are introduced into the columns by end plates. The bending moments are transferred into the columns by reinforcement and contact plates at the top flange and by contact in combination with welded plates at the bottom flange. In comparison with the paint unit in Rüsselsheim, the composite columns had the advantage of increasing the horizontal stiffness of the frame significantly. To avoid the high costs for the formwork of completely concrete encased sections, partially encased sections are used for the columns. Additionally this has the advantage that new installations can be fixed at the free steel flanges without difficulties. In Germany it was the first time that a mixed system with composite columns, composite girders and steel trusses was designed as a composite sway frame. The design of this type of framing is not covered by Eurocode 4 and the national German codes. In a first trial calculation an elastic calculation was carried out with an effective cracked stiffness for the columns. The effect of cracking of concrete in the main girders was taken into account by reducing the stiffness at internal supports to the stiffness of the steel section consisting of structural steel and reinforcement. Effects of creep and shrinkage were taken into account by use of different modular ratios for permanent actions, shrinkage and hyperstatic effects developing in time. For the standard frames the second order effects lead to an increase of bending moments up to 25%, which is significantly lower than the second order effects of the steel frame of Opel Rüsselsheim. In a second step, a non-linear calculation was carried out for critical load arrangements, taking into account cracking of concrete and tension stiffening effects. These calculations have shown that the simplified method gives safe results.

Figure 4 shows the material distribution of the main girders of the body unit of Porsche and the paint unit of Opel. It is noticeable that the number of changes of cross-sections of the flanges and the web of the Opel beam is significantly reduced. As explained above, in the design of the girder of Porsche the concrete encasement and the reinforcement were taken into account only for fire resistance. For the Opel girder the concrete encasement and the reinforcement were used to improve the bending resistance for normal temperature design as well as fire design. For economical reasons, instead of altering the flanges of the structural steel section, additional reinforcement in the concrete encasement was provided. Table 1 gives a calculation example for a typical main girder with concrete encasement and additional reinforcement, and in comparison for a girder with conventional fire protection by encasement with fire resistance boards, based on average unit prices of the year 1996. The comparison demonstrates that the concrete encased beam is more economical and has the additional advantages described above and has as well a significantly higher flexural stiffness.

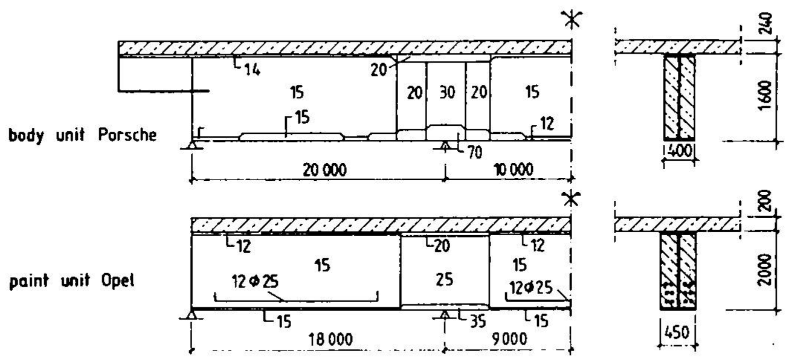
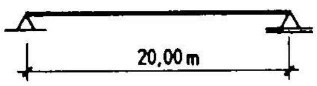
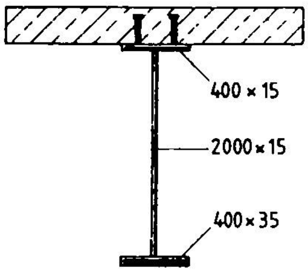
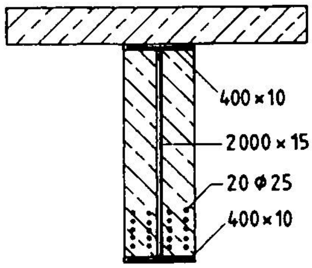


Fig. 4 Material distribution of the main girders of the body unit of Porsche and the paint unit of Opel in Rüsselsheim

Table 1 Comparison of costs for typical main girder

system	structural steel 7,85 tons		structural steel 6 tons	
				
	unit price in ECU	total costs ECU	unit price in ECU	total costs ECU
structural steel (wage costs)	500 ECU / ton	3925,0	460 ECU / ton	2760,0
structural steel (material)	500 ECU / ton	3925,0	500 ECU / ton	3000,0
headed studs (top flange)	2,5 ECU/stud	750,0	2,5 ECU/stud	750,0
erection costs/ton structural steel	200 ECU/ton	1570,0	300 ECU/ton	1800,0
fire resistance boards ($88m^2$)	50 ECU/m^2	4400,0	-	-
concrete encasement ($15{,}25m^3$)	-		250 ECU/m^3	3800,0
studs concrete encasement	-		2,2 ECU/stud	660,0
reinforcement concrete encasement 1,4 to.	-		700 ECU/ton	980,0
total costs for the beam	**14570.0 ECU**		**13750.0 ECU**	

In contrast to the body unit of Porsche the span of the composite slabs is increased to 3,0 metres for the Opel paint unit. Propping of the sheeting is normally necessary for a slab with Holorib sheeting and a thickness of 200 mm. The slabs were poured in two layers, in order to avoid the high costs of propping in industrial buildings with a height between floors of more than 10 m. The thickness of the first layer resulted from the bending resistance of sheeting acting as formwork without props. After hardening of the first layer the composite slab was capable to resist the remaining concrete for the required slab thickness of 200 mm. To achieve a sufficient longitudinal shear resistance between the two concrete layers, additional shear reinforcement had to be provided.

3 Office, Production and Storage Building of Siemens in Berlin

Figure 5 shows a multi-storey building for Siemens in Berlin, erected in 1993 /3/, /4/. Maximum flexibility was required by the client with regard to the usability of different parts as office, production and storage areas. The building rises five stories on an area of 81 x 81 meters with a column grid of 14.8 x 10.8 m. The slabs with a span of 3.6 m and a depth of 20 cm are composite with Holorib steel sheeting. All beams with a depth between 450 mm and 900 mm are continuous with concrete encasement and additional reinforcement taken into account in the design at normal temperature and for fire resistance. Slabs and beams had to be designed for traffic loads up to 30 kN/m^2. In the areas with a height between floors of 11.5 m the concrete was poured in two layers to avoid propping of the sheets. In all other areas propped construction was preferred for the sheeting. The columns are partially concrete encased I-sections and concrete filled tubes. In the centre of the building, a big composite grillage with an integrated composite ring beam is located for a domelight. The building of Siemens is an good example how the architectural requirements showing the steel members can be combined with high fire resistance and high load bearing capacity of the structure.

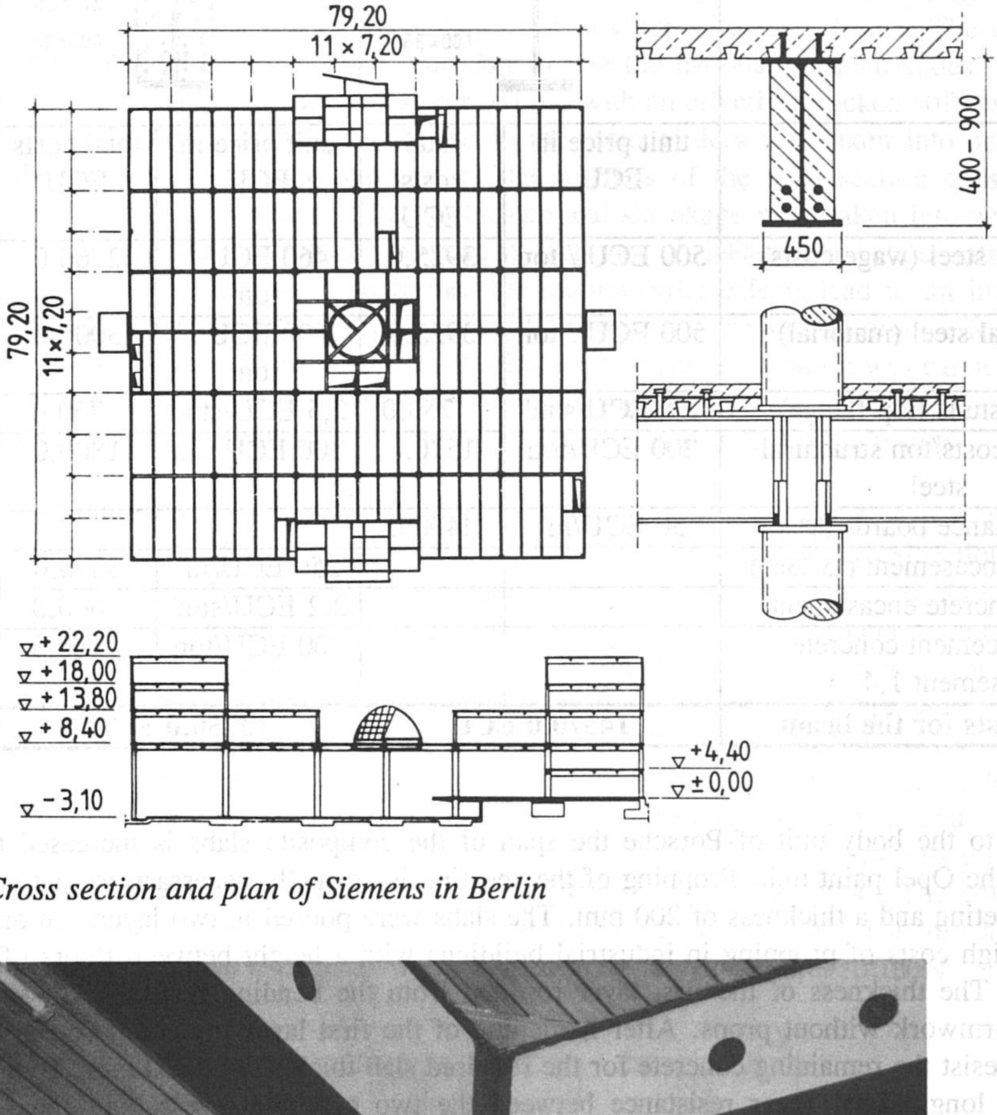


Fig. 5 Cross section and plan of Siemens in Berlin

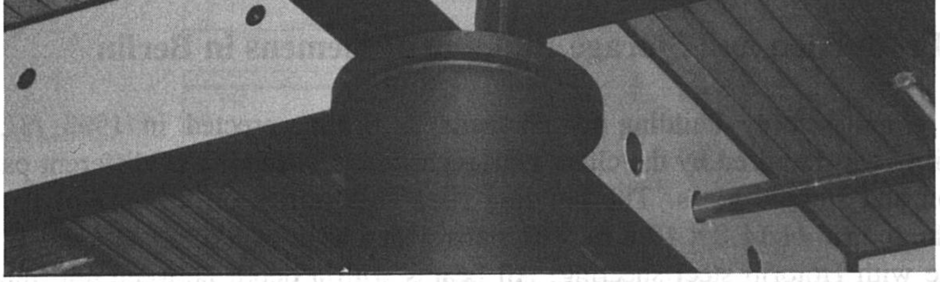


Fig. 6 Architectural shaped beam-column connection

4 Extension of the Airport in Hannover

Figure 7 shows the extension of the airport in Hannover under construction in 1997. The two storey building has a triangular plan with a side length of approximately 144 m. It is stabilised for horizontal action by concrete cores. Most columns are concrete filled steel tubes. The beams are concrete encased welded I-sections with an extremely high ratio of reinforcement. Between the flanges, up to 20 bars with a diameter of 28 mm are provided. As explained above, this type of beam is economical because the unit prices for reinforcement are significantly lower than those for structural steel and because the bending resistance at normal temperatures and not the fire resistance governs the design. For this special type of composite beam, tests were carried out to ensure that the total reinforcement of the concrete encasement can be taken into account for the plastic bending resistance, where there is only 50% of full shear connection. Furthermore the crack formation, the crack width and the deformation behaviour of the beams were checked because of the special public interest in the building. The test results have shown, that for beams predominately loaded by point loads the design rules in Eurocode 4 regarding the minimum degree of shear connection are very conservative. In addition to the main reinforcement of 28 mm bars, small-diameter bars near the surface had to be provided, to ensure a sufficient distribution of cracks in the concrete encasement in order to develop the full plastic bending resistance and to avoid early failure of the main reinforcement at single big cracks.

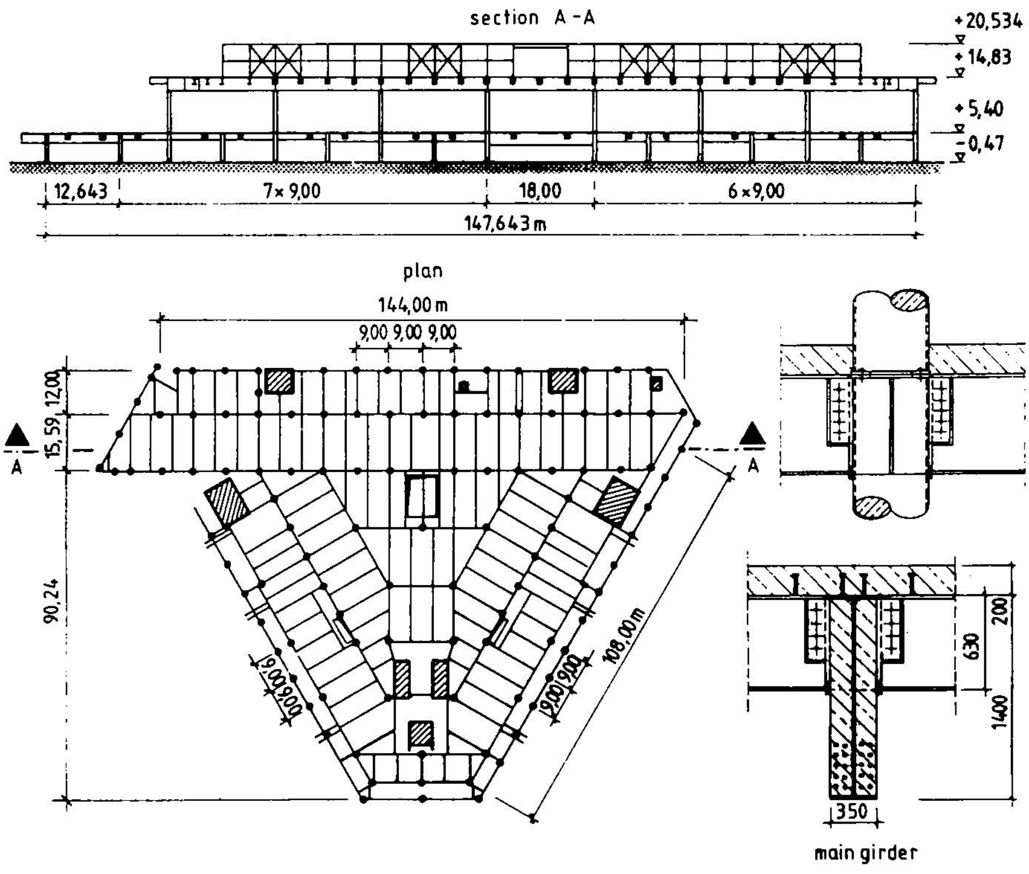


Fig. 7 Plan of the extension of the airport of Hannover

5 High-rise Buildings

5.1 General

In 1997 two very interesting building projects will be finished in Düsseldorf and Frankfurt. The Stadttor (Towngate) in Düsseldorf /5/ and the Commerzbank tower in Frankfurt /6/. Both high rise buildings use a large variety of different composite members with sophisticated load-bearing structures.

5.2 The Towngate Düsseldorf

The building with a rhomboid plan consists of two 16 storey office towers connected by horizontal truss members extending through the three top floors (Fig. 8). The horizontal stability is realised by small concrete cores and mainly by three portal frames consisting of composite truss-columns and the horizontal truss members. The three portal truss-frames form a z-shaped system in the plan. The trusses are made by concrete filled steel tubes where the horizontal and diagonal members above the fourth floor are without concrete. Composite slabs in combination with concrete encased composite main beams and secondary composite beams without concrete encasement are used for the vertical loads. With regard to the required fire resistance Class R90 cementious coatings or fire resistance boards are employed for the secondary beams. The fire resistance of the columns is ensured by concrete and additional reinforcement without any further protection of the steel tubes.

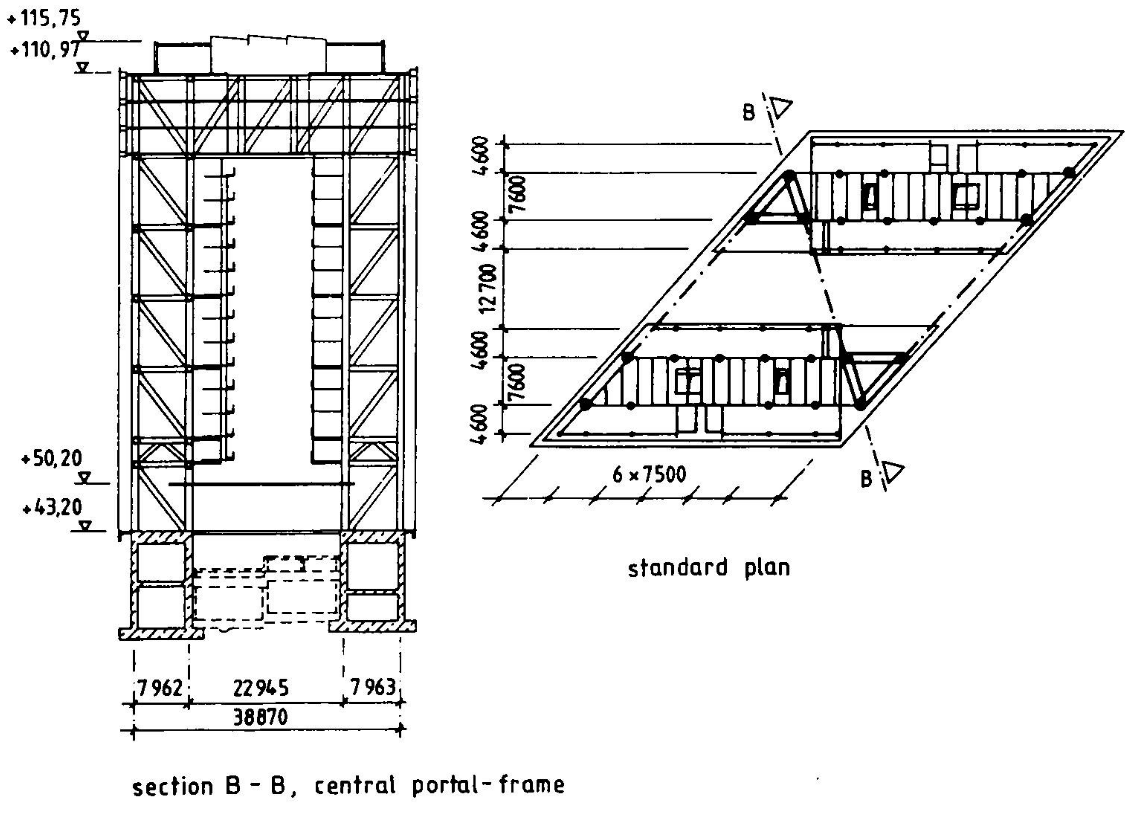


Fig. 8 Plan and section of the Towngate Düsseldorf

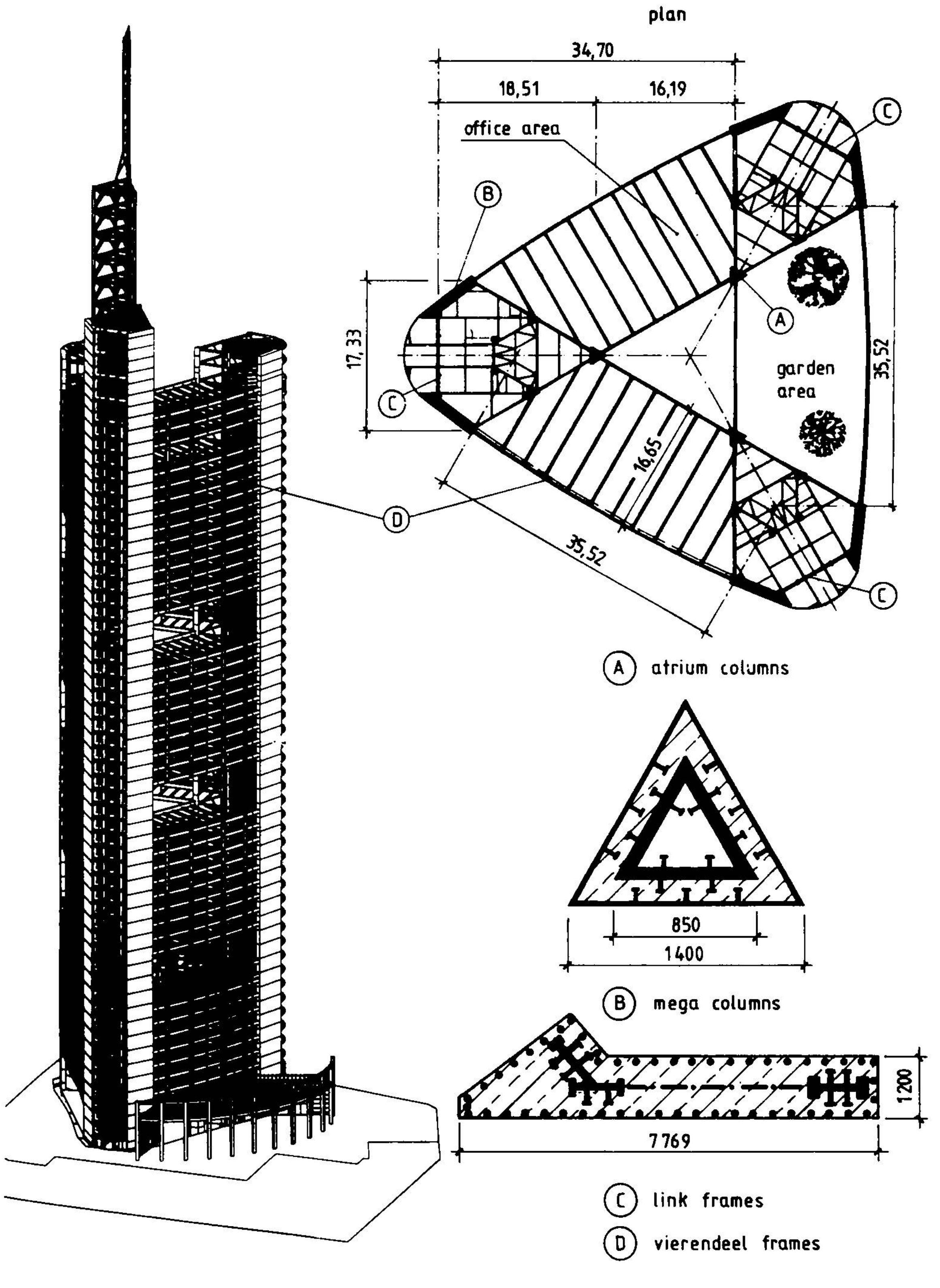


Fig. 9 Commerzbank Headquarters Frankfurt/Main

5.3 Commerzbank Headquarters in Frankfurt /Main

The new building of the Commerzbank is the most interesting project in Germany at the present time. With a overall height of 298.74 m, including antenna, 63 floors and an effective area of 52700 m^2, the building will give room for 2400 employees of the Commerzbank. The plan of the building has the shape of an equilateral triangle with rounded corners and slightly curved sides with length of approximately 60 m (see Fig. 9). Three cores at the corners, ending at different heights, contain stair cases, elevators, adjoining corridors and the installation for the building. The cores are connected by office areas where the standard floors consist of two office areas. The third side is kept free over 4 storeys and contains a garden area. This standard floor plan is turned by 120 degree every four storeys. For the horizontal loads, a three dimensional structure (consisting of the mega-columns connected by link frames, the vierendeel frames located in the facade, and the composite slab floors) forms a tube, which is fixed in the foundation. All other columns inside the cores and the atrium columns are nominally pinned at both ends and do not contribute to the horizontal stability of the building. The composite floors consist of Super-Holorib sheeting in combination with light weight concrete with a density class of 20. In the office area composite beams with welded sections and large web openings span between the atrium beams to the outside vierendeel steel frames. Within the cores mixed systems with steel and composite beams are used. The required fire resistance Class R120 for the beams was achieved by fire resistance boards. The atrium columns are designed as composite columns with an inner and outer equilateral-triangle steel section with steel grades S460 and S355. Composite action is achieved by headed stud shear connectors. Because this type of section is not covered by Eurocode 4, an elasto-plastic design had to be carried out. The design for fire resistance is based on a reduced effective section with decreased material properties. The mega columns in the edges of the plan consist of fully concrete encased steel trusses forming a composite column together with concrete and additional reinforcement. All other columns are designed as steel members.

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Slim Floor Construction: Why?

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Summary

Slim floor construction is being used increasingly in Europe. Several construction systems have been developed and their characteristics are described on behalf of some buildings which just have been completed.

1. Historical Background

The architects and owner wish for greater flexibility in multi-storey construction has led to more frequent use of slim floors. The main characteristic of this form of construction is a shallow floor in which beams and slab elements are integrated within the same depth.

In fact the principle of the slim floor exists at least since the middle of last century. In 1845 timber floors were replaced in Great Britain by stone arches comprising integrated iron beams. A typical development consisted in the so-called "Prussian Cap Floor", shown in figure 1.

By the end of the nineteenth century standard rolled sections were used as integrated within the concrete slabs. However as the span of the concrete was still quite small, the distance between the steel beams was also quite reduced.

New developments in steel and concrete construction have favoured a real "Renaissance" of slim floor construction since 1975. According to Dr. Lars Wallin of the Swedish Institute of Steel Construction [1], "One effective way to reduce the total floor construction height is to support the floor elements on the bottom flanges of the floor beams. For that purpose a low, welded floor beam with a wide bottom flange and a narrow top flange has been developed. The selected beam height should be approximately equal to or somewhat lower than the floor element thickness. Hence the floor construction height can be reduced quite substantially. This type of floor beam can also be used with composite action through welded studs and with continuity to further reduce the beam height."

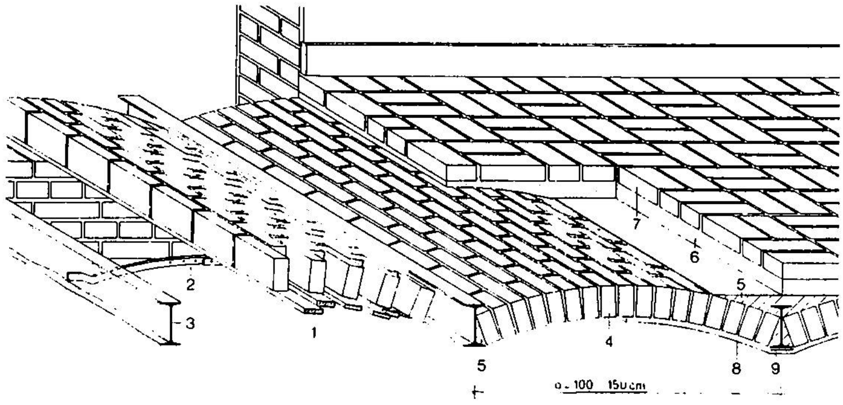


Fig. 1 Prussian cap floor

1 Shutter	*4 Wall bricks*	*7 Floor bricks*
2 Guide	*5 Concrete*	*8 Parget*
3 Floor beam	*6 Sand*	*9 Parget support*

Consequently a particular steel frame system characterized by slim floors and integrated fire resistance has become the dominant system for many buildings in Nordic Countries. Corresponding steel beams are shown in figure 2a, b and c. Slightly different integrated floor beams are now being used in Continental Europe and in Great Britain according to figures 2d and 2e, and to chapters 2.1 and 2.3 [2].

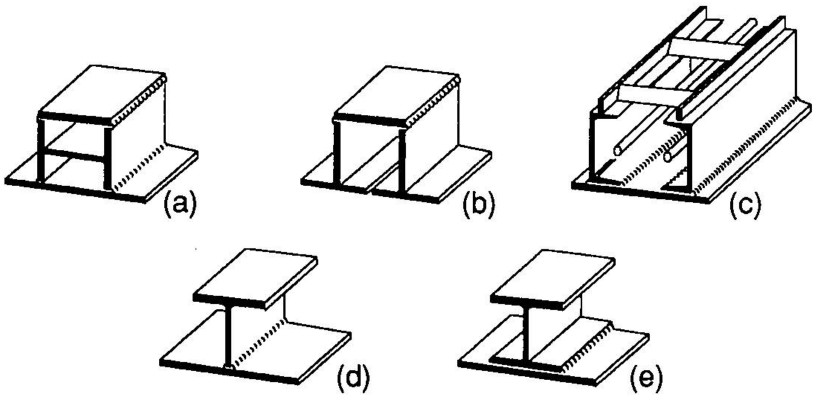


Fig. 2 Slim floor beams in the Nordic countries (a,b,c) and in Continental Europe and Great Britain i.e. IFB (d) and SFB (e).

2. Variety of Applied Systems

Various slim floor systems are presently available with different combinations of steel beams and concrete or composite slabs such as

- integrated floor beam IFB with hollow core prestressed units HCU,
- rolled profiles with in-situ normal weight concrete, the S+V system,
- slim floor beam SFB with deep decking and in-situ light weight 7concrete, the "Fast Track Slim floor",
- rolled profiles with deep decking and in-situ normal weight concrete, the so-called "Additional Slab".

2.1 IFB Slim Floor

The slim floor based on the integrated floor beam IFB was developed by PROFIL-ARBED [2,3,4,5] since 1991. It represents a part of a dry construction system based on steel columns, one directional steel beams and prestressed hollow core units (see figure 3).

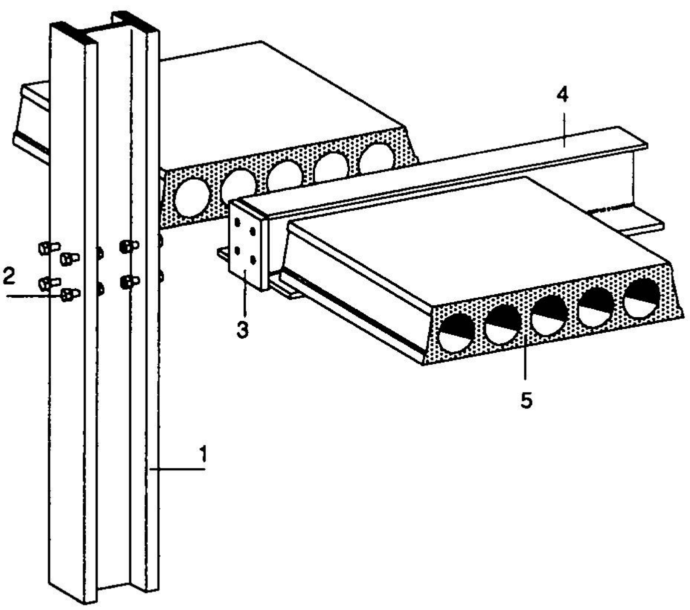


Fig. 3 Typical IFB slim floor system

1 Steel column
2 Bolts
3 End plate welded to beam
4. Steel beam IFB
5 Hollow core unit

The IFB concept is characterized by the following items described in figure 4:
- the steel beam is composed of a lower steel plate welded to the web of the half of a rolled section,
- various steel profiles and steel grades with a yield point of 235 up to 460 N/mm^2 are available,
- the integrated floor beam may be designed as simply supported or as a continuous beam,
- the void between hollow core units and the web of the steel profile is filled with in-situ concrete,
- it is strongly recommended to supply an in-situ topping with reinforced concrete in order to improve the transversal load distribution between hollow core elements. This structural topping is also increasing the shear resistance of hollow core units [6],
- a similar effect is produced by filling the cores at each end of the slab units,
- to ensure sufficient shear resistance to hollow core units during exposure to fire, it is recommended to provide complementary stirrups [7],
- complementary reinforcing bars, parallel to the steel beam, guarantee its ISO-fire resistance up to 120 minutes, whereas the lower steel flange may remain unprotected [8].

As a consequence the floor slab, supported on the bottom flange of the steel beam, presents from beneath a straight and clean ceiling surface.

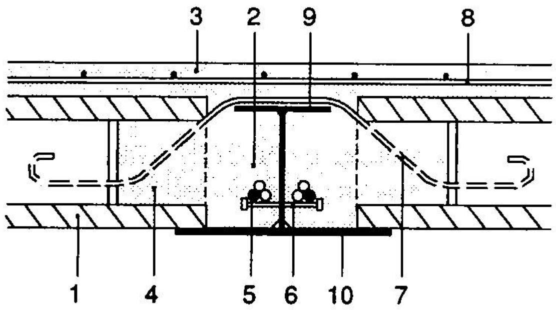


Fig. 4 IFB cross section

1 Prestressed hollow core unit
2-3 In-situ concrete
4 Core filling with concrete
5 Reinforcing bars
6 Welded studs
7 Stirrups
8 Slab reinforcement
9 Half of rolled profile
10 Welded bottom plate

2.2 S+V Slim Floor

This slim floor based on rolled profiles with various shapes was developed by STAHL & VERBUNDBAU GmbH [2]. It represents a part of a construction system based on steel or composite columns, steel beams and in-situ reinforced concrete slab (see figure 5).

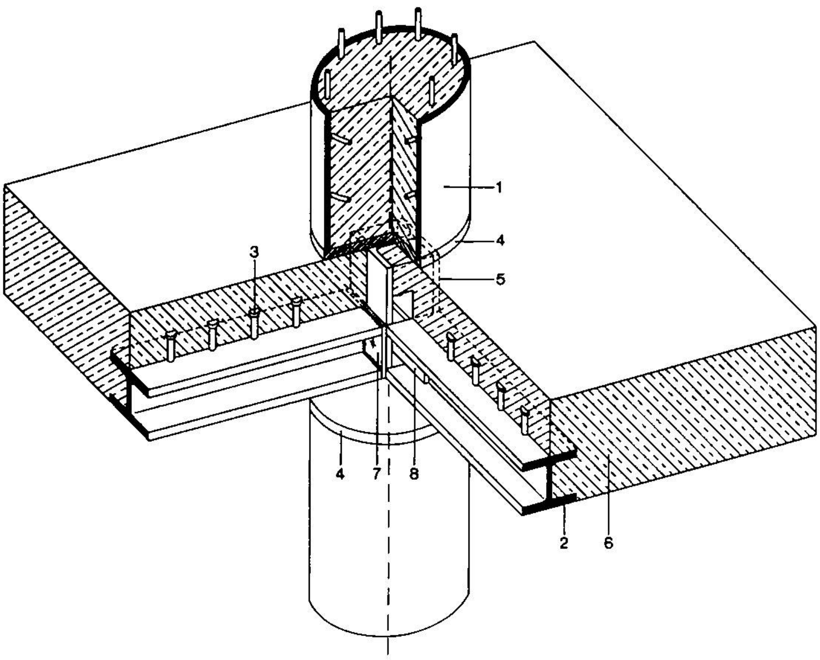


Fig. 5 Typical S+V floor system
1 Composite hollow section column
2 Slim floor steel beam
3 Shear studs
4 End plates of columns
5 Steel profile
6 In-situ reinforced concrete
7 End plate of beam
8 Shear flat for 2

The S+V concept is characterized by the following items described in figure 6:

- the steel beam is composed of rolled profiles or half sections with or without a steel plate welded to the free web end,
- various steel profiles and steel grades with a yield point of 235 up to 460 N/mm^2 are available,
- the steel beams may be designed as simply supported or as continuous beams,
- during in-situ concreting of the reinforced slabs, concrete may be supported either by a timber shutter, prefabricated filigree concrete decks or profiled steel sheets,
- the steel profile, encased in concrete with the exception of the lower steel flange, behaves as a composite cross-section, also thanks to the shear studs welded on the profile,
- this cross-section may be designed according to ENV1994-1-1 for normal temperature conditions [9] and following ENV1994-1-2 for fire exposure [8]; the lower steel flange may remain unprotected up to the ISO requirement of R120,
- this slim floor cast in-situ has an assured monolithic behaviour, useful in certain design situations when f.i. large openings have been foreseen in the floor or for special loadings due f.i. to earthquake shocks !

This slim floor, similarly as the previous one, presents from beneath a straight and clean ceiling surface.

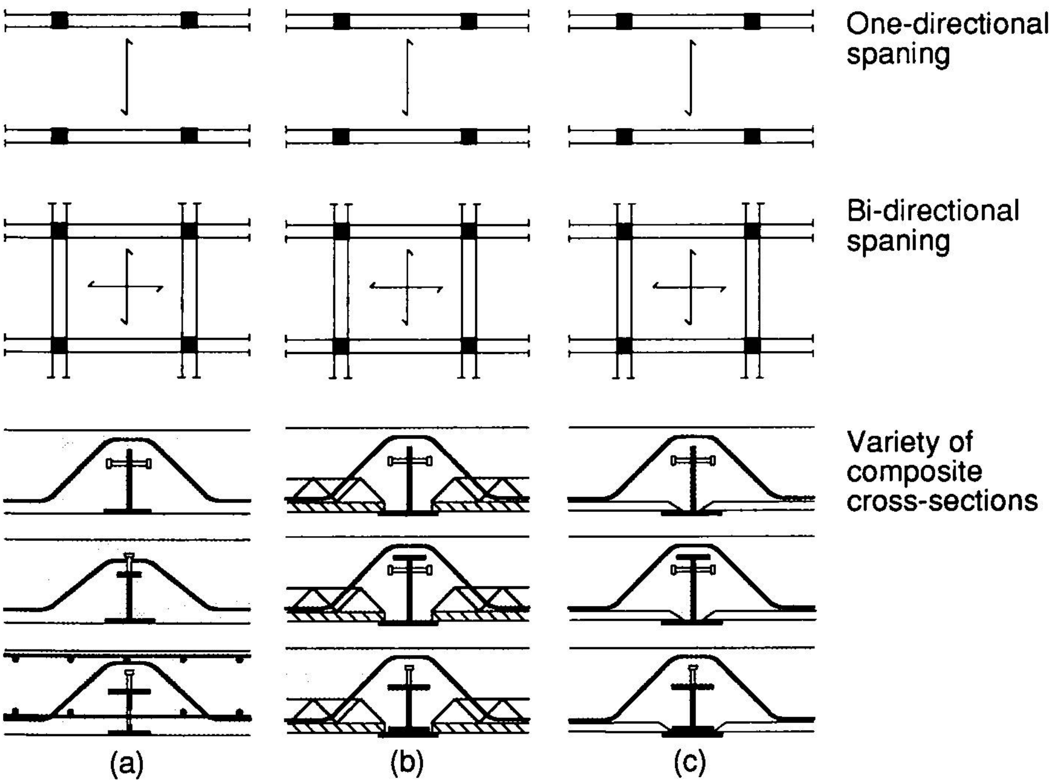


Fig. 6 S+V cross sections based either on a timber shutter (a), prefabricated filigree concrete decks (b) or profiled steel sheets (c)

2.3 Fast Track Slim Floor

This slim floor based on the SFB beam was developed by BRITISH STEEL [2,3,4]. The corresponding construction system is based on any steel or composite column type, steel beams with enlarged lower flange supporting the deep decking, and in-situ reinforced light weight concrete (see figure 7).

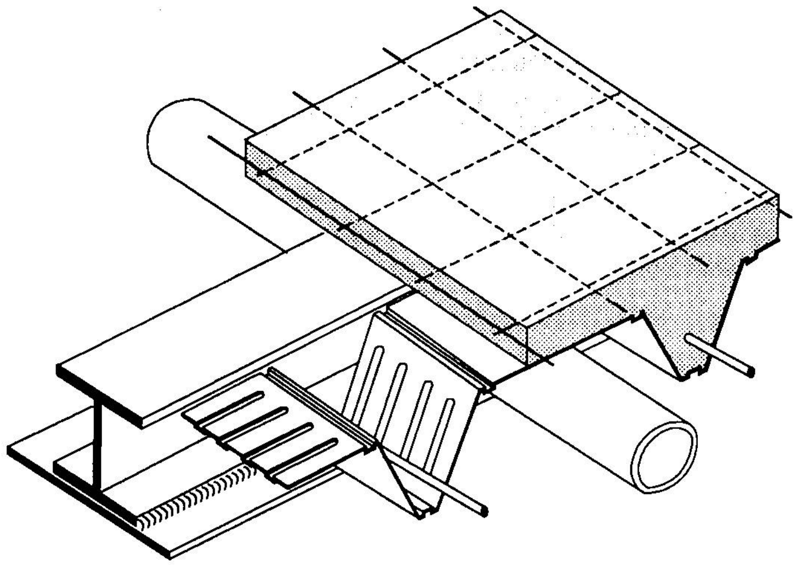


Fig. 7 Typical Fast Track Slim Floor

The Fast Track Slim Floor is characterized by the following aspects:

- the steel beam is composed of rolled profiles reinforced on the lower flange by an enlarged and welded steel plate,
- various steel sections and steel grades up to a yield point of 355 N/mm^2 are used,
- reinforcing bars are foreseen inside and on top of the deep decking,
- light weight concrete cast over the whole floor constitutes a concrete lattice in which the steel beam is fully integrated,
- composite actions is activated by welding shear studs on top of the steel beams and by the embossments of the deep decking,
- large openings may be realized through the webs of SFB beams, permitting to integrate all types of pipes beneath the deep decking but still localized within the height of the slim floor,
- this slim floor system may be designed according to ENV1994-1-1 and ENV1994-1-2 [9,8]; specific design rules have been confirmed by full scale testing in the Netherlands as well for normal temperature conditions as for fire exposure [10,11,12,13].

2.4 Additional Slab-Slim Floor

This slim floor based on rolled profiles of any shape was developed by HOESCH SIEGERLANDWERKE GmbH [2]. The corresponding construction system is based on any steel or composite column type, steel or composite beams fabricated out of rolled profiles with shear studs and bearing blocks welded on to the upper flanges which are supporting the deep decking, and in-situ reinforced normal weight concrete (see figure 8).

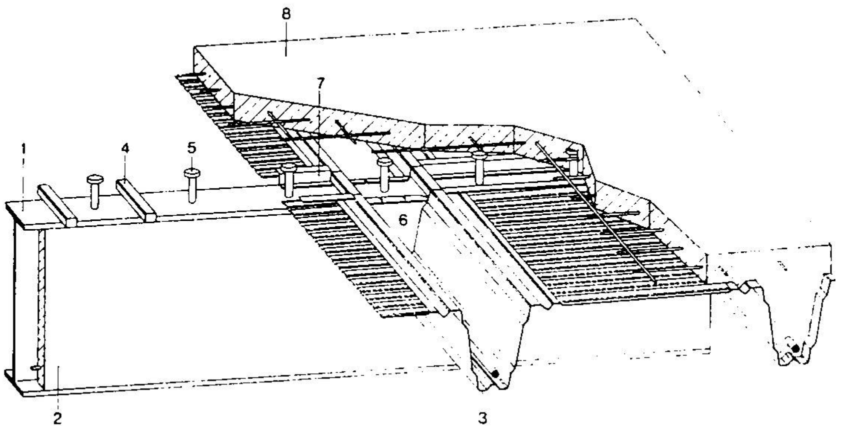


Fig. 8 Typical Additional Slab-Slim Floor
1 Composite beam — *5 Shear stud*
2 Partially encased concrete — *6 PVC end plate*
3 Deep decking steel sheet — *7 Z shaped stanching profile*
4 Bearing block — *8 Reinforced concrete lattice*

In fact this "Additional Slab" is not, generally spoken, a slim floor, as the steel beam may be higher than the deep decking. This floor system is characterized by the following aspects:

- the steel beam is composed of rolled profiles of any shape and height and any yield point up to 460 N/mm^2,
- reinforcing bars are foreseen inside and on top of the deep decking, supported by special bearing blocks welded on top of the steel beams,
- normal weight concrete cast over the whole floor constitutes a concrete lattice,
- composite action is guaranteed by welding shear studs on top of the steel beams and by the embossments of the steel decking,
- this floor system may be designed according to ENV 1994-1-1 and ENV 1994-1-2 [9,8]; the ISO-fire requirement R120 may be fulfilled.

3. General Design Recommendations

Design recommendations may be split in general and specific principles and rules, function of the various slim floor systems described in chapter 2, whereas general recommendations should be issued concerning global fire safety considerations.

3.1 General Design Principles

It is in the interest of construction in general and of steel construction in particular to conceive safe slim floors for normal temperature conditions and for the fire situation. Consequently it should be checked that design rules, both for bending resistance and shear resistance, have been calibrated on the basis of thorough calculations and realistic tests.

It may be assumed that these requirements are fulfilled for the bulk of slim floor systems given before and designed according to ENV 1994-1-1 and ENV 1994-1-2 [9,8].

3.2 Specific Design Rules for Slim Floors with HCU

The following special features shall be considered for normal temperature conditions and for the fire situation.

3.2.1 Normal temperature design
Shear resistance of hollow core units, HCU, should be established as a function of the flexibility of the corresponding supports. Investigations and research, undertaken in Nordic countries since 1990, clearly indicate that the shear resistance of HCU elements decreases when the flexibility of the supporting beams increases [14,15].
Furthermore it is considered that in-situ structural topping may contribute in a significant way to improve the shear resistance of hollow core units [4,6].
Finally it would be useful to study the "General Shear Design Method" established by COLLINS & MITCHELL [16]. This so-called unified method is presented for the shear design of both prestressed concrete members and nonprestressed concrete members. The method can treat members subjected to axial tension or axial compression and treats members with and without web reinforcement. In fact the normal cracked concrete contribution V^c_{Rd} is added to the reinforcement contribution V^s_{Rd} and to the effect of prestressing V^p_{Rd}, so that the shear resistance follows from:

$$V_{Rd} = V^c_{Rd} + V^s_{Rd} + V^p_{Rd} \tag{1}$$

3.2.2 Structural Fire Design
It appears that the structural fire design of the steel beams composing slim floors may be performed according to well established procedures [8]. However when analysing hollow core units, HCU's, for the fire situation, it becomes quite fastidious to get hold of a complete set of design rules including bending and shear resistance. Whereas DIN 4102, Teil 4, gives constructional requirements related to bending resistance [17], other regulation codes give some guidance only concerning the shear resistance.

In chapter 2.7.3.2 (ii) of the FIP RECOMMENDATIONS [7] it is written that *"The shear capacity of hollow core units during exposure to fire is affected by (a) the increase of the tensile stresses in the webs due to the temperature gradient etc."* and in chapter 4.5, Rule (4) of ENV 1992-1-2 [18] it is declared that *".... special consideration should be given when tensile stresses are caused by non-linear temperature distributions (e.g. voided slabs, thick beams, etc.). A reduction in shear strength should be taken equivalent to these tensile stresses"*. Unfortunately no quantified design rule is given permitting to check the shear resistance in the fire situation, $V_{Rd,\theta}$, of hollow core units.
For that reason several slim floor fire tests were performed in France and in Switzerland [19,20], clearly testifying the beneficial effect when providing a structural topping and stirrups (see figure 4). Furthermore a first attempt was done to quantify, in connection to finite element calculations, the shear resistance of HCU's [21,22]. In that respect the procedure explained in [16] was adapted to the fire situation

$$V_{Rd,\theta} = V^c_{Rd,\theta} + V^s_{Rd,\theta} + V^p_{Rd,\theta} \tag{2}$$

Consequently it is recognized, that slim floors based on HCU's constitute a quite complex construction system with steel, concrete and prestressed components. This situation requires multi-disciplinary research based on a scientifically sound background and supported by a set of realistic fire tests.

3.3 Global Fire Safety Considerations

Developments under way since 1985 in the field of fire safety engineering permit nowadays to take advantage of the new structural fire design standards of CEN. Indeed global structural analysis may be performed on the entire structure, the combination rule for actions during fire shall be considered and the evolution of a natural fire may be assumed ! This realistic structural fire design may even be improved by considering the effect of active fire safety and fire fighting measures. Consequently this produces real safety for people, f.i. by adequate partitioning, by sufficient escape routes and proper smoke venting [23,24,25].

4. Specific Architectural Outcomes

Our intention is not to discuss principles of architecture, like those concerning the so-called architecture of freedom. But it is worthwhile to remember MIES VAN DER ROHE who said: *"A clear structure is the backbone in it all and makes the free plan possible".*

From that point of view slim floor construction is quite attractive as
- it allows an integrated layout for services and ventilation ducts (see 2.3 and 2.4)
- it permits in general a reduced floor to floor depth,
- it leads to a clear and straight ceiling surface so that to make the structure of the floor visible and so that a shifting of inner partition walls is always possible (see 2.1 and 2.2).

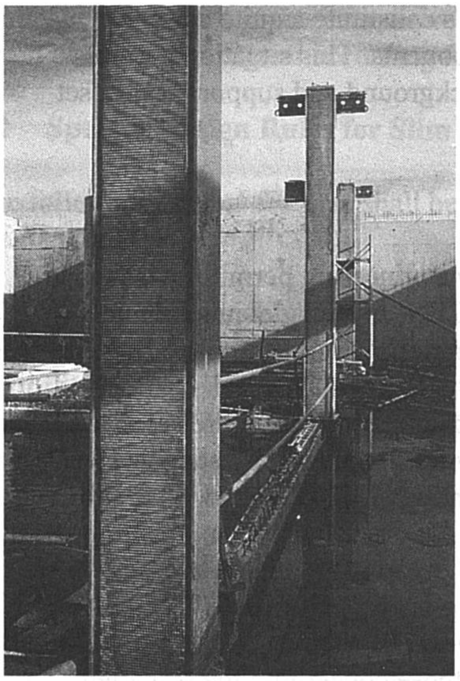
In order to illustrate architectural advantages of slim floors, the following buildings are shortly described hereafter.

4.1 Ecole Nationale des Ponts et Chaussées, Marne-la-Vallée, France, 1994-96

This ENPC high school building with a total clear surface of 31.200 m^2 has been erected in the "City Descartes" in the East of Paris. The bulk of floors is conceived as IFB slim floors [26].

The following special features may be underlined:

- composite columns with partially encased steel profiles (HE300AA), designed according to ENV 1994-1-2, were erected in elements covering two levels. The grooved concrete and the steel flanges are remaining visible (see figure 9),
- the integrated floor beams IFB are composed of the half of the rolled profile HP400 on top of which the steel plate 140x40 mm was welded. Furthermore shear studs were welded to this upper flange in order to create composite beams (see figure 10),
- a structural reinforced topping of 8 cm thickness was put on top of the hollow core units HCU, so that the total floor thickness is 25 cm.



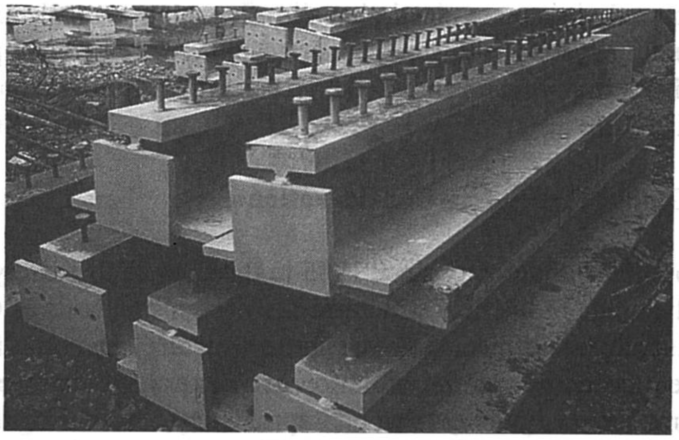


Fig. 10 ENPC, IFB beams with shear studs

Fig. 9 ENPC, Composite columns

4.2 PROFILARBED Office Building, Esch-sur-Alzette, Luxembourg, 1991-93

This AOB office building is composed of two wings with nine levels and a total volume of 61.000 m^3. The global fire safety has been designed according to 3.3, so that the steel structure remains visible in the entire building. Floors are conceived as IFB slim floors [23,24].

The following special aspects could be underlined:
- the integrated floor beams IFB are composed of the half of the rolled profile IPEA500 below which a steel plate 420x10 mm was welded,
- a structural reinforced topping of 10 cm thickness was cast on top of the hollow core units HCU, in order to guarantee sufficient ductility (see figure 11),
- this building shows in a striking way the lightness and beauty of a steel frame when a global fire safety concept is used together with slim floors (see figure 12).

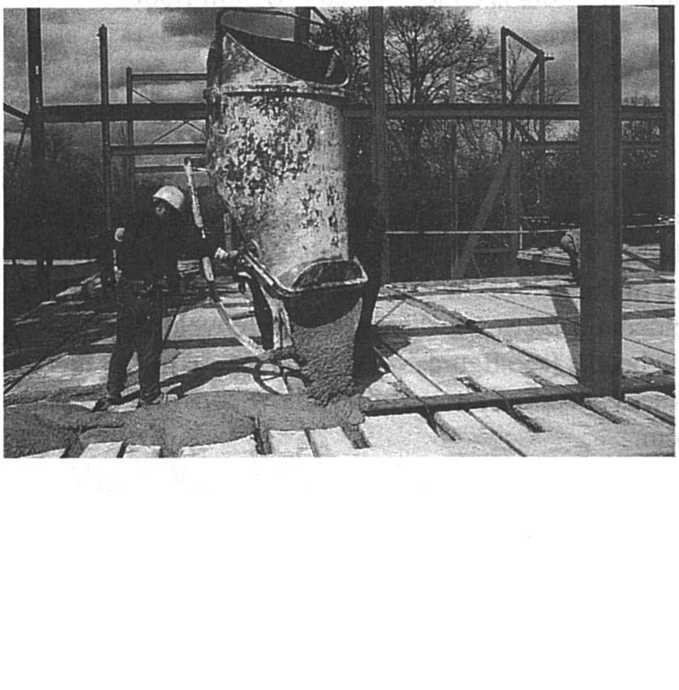


Fig. 11 AOB, IFB slim floor based on hollow core units reinforced by stirrups at supports and by a structural topping

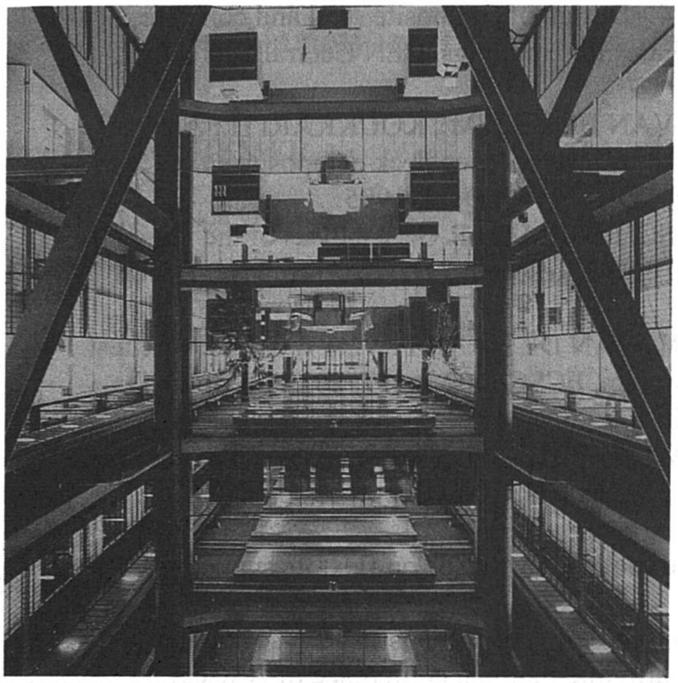


Fig. 12 AOB, View inside the building atrium on slim floors and visible steel structure

5. Conclusion

Multi-disciplinary and correctly guided research is needed to develop fully transparent design rules in relation to slim floors. In that case slim floors in general, and those based on hollow core prestressed units in particular, may be considered as the most promising construction models in the future.

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Variety of Composite Bridge Construction

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Summary

The construction of composite bridges is rising and numerous publications give an outline of the success met by this type of bridge. Everywhere in the world but with developments differing according to the countries, innovations and improvements come up. Girder and truss bridges, arch and bowstring bridges and, of course, cable-stayed bridges, all types of bridges allow to benefit from the steel concrete composite technique to design high quality bridges, so much in terms of economy as of aesthetics and durability.

1. Introduction

For the last few years, the construction of composite bridges has known a growing success worldwide, with developments of course differing according to the countries.

This wide success can be explained by the improvements and innovations that occurred these last two decades in many industrialized countries.

Most often, they can be :
- either a progress in the use of materials, which characteristics have been improved (high resistance weldable steel, plates with variable thickness),
- either more daring designs, permitted by a better knowledge of structural mechanics (unstiffened webs),
- or improvements of techniques and construction methods, facilitated by the development of handling means and by the industrialization of the tools in the workshops.

The innovations are also due to the abilities of the composite structures to solve, with elegance, lightness and economy, problems that seemed unadapted to full concrete bridges or too expensive for steel bridges with an orthotropic slab.

Less spectacular innovations, although their importance will appear as the years go by, are related to the improvement of the execution, to the care in the design of the details, to the quality of the materials and, most of all, to the aesthetical quality of the bridges.

Presently, there is no doubt left that the innovations have first to be searched to increase the overall quality of the bridges : technical quality and durability, but also aesthetical quality and care of the environmental integration.

The future of the composite bridges is full of promises; beyond its numerous qualities (lightness, fast and secure realization, weak sensitivity to differential settlements), they have a henceforth major advantage, they are durable. The promises are even greater if the effort is made to design them in accordance with their specificity and not as bad copies of steel or concrete bridges.

The evolution of the calculation methods on a theoretic level (progress in the stability, torsion and fatigue domains) and the computer means allows to simplify the structure. The number of stabilizing elements, such as the stiffeners, cross beams and wind-bracings, decreases and they are sometimes even completely suppressed.

This tendency towards a greater simplicity is even reinforced by economical requirements. Facing the increasing costs of the manpower and the rather stagnant cost of the steel, the bridge builders add even more to this simplicity to decrease the working time.

The evolution of the knowledges, the calculation means and the manpower costs cannot explain by itself the totality of this change. Twenty or twenty-five years ago, the first criterion to choose a variant for a bridge, either a composite or prestressed concrete one, was its price; other criterions, such as durability or aesthetics, were often further relegated.

This order in the appreciation criterions has led to quite a few disappointments. Some bridges, after a few years only, had to be heavily repaired. Other ones, rather inaesthetical, have been severely criticized among the population. Therefore, the hierarchy of these criterions has evolved. Presently, we think that price, durability and aesthetics have the same importance.

2. The principles

The initial justification of the composite structure is the idea to use at best the own qualities of the materials :

- the concrete, for its good compressive strength and its low cost per volume unit,
- the steel, for its high tensile strength.

The typical example is obviously the cable-stayed bridge. We all know that the pylon and the deck of modern multi-hangers bridges are mainly compressed, along with weak bending moments, as the hangers are obviously only tense.

The cross-section of this same box-girder cable-stayed bridge with a central layer of hangers, shows steel inner tense struts and a concrete compressed box-girder. These are the leading principle for the WANDRE and BEN-AHIN bridges in Belgium, for the ELORN bridge in France...

The second justification, that explains the quality of the composite beam, comes from an economical analysis of the various constituents of the resistant structure of a bridge deck. The price of a full metallic bridge is burdened by the exorbitant cost of the orthotropic slab that, in spite of a very light weight, asks a very important working time.

On the other hand, in concrete bridges, the webs of the girders are usually too thick on a matter of resistance, as the minimal thickness is usually bounded to technologic requirements such as the presence of prestressing cables and the concreting operations.

The third justification is the possibility to use steel for its lightness in areas where the weight has a baleful influence and to place the concrete where the dead load of the structure, either has little influence, or is favorable to the stability. Numerous examples have been built on basis of this principle these last years, the CHEVIRE and DEL MILENARIO bridges and, of course, the most famous one, the NORMANDY cable-stayed bridge.

3. The deck slab

For more than 20 years, bridges have been built with composite girders, continuous on their bearings, where the concrete deck slab has to suffer from severe tensile stresses. The experience shows that these bridges behave very well as the tense slab is either prestressed enough or largely reinforced and, of course, as used is made of a high quality concrete.

Unfortunately, many of those continuous composite structures have insufficient reinforcement steel and know longevity problems, particularly in countries where de-icing salts are used. The problems related to the durability of the deck slab are similar to the ones of the full concrete bridges; they need a special care and a extremely compact concrete.

4. The bridges

The different kinds of bridges will reviewed about and, in each case, the large variety of solutions able to be brought by the composite construction will be pointed out.

4.1 Girder bridges

The last two decades have shown a wide simplification of the composite structures, for the full web girder and truss girder bridges.

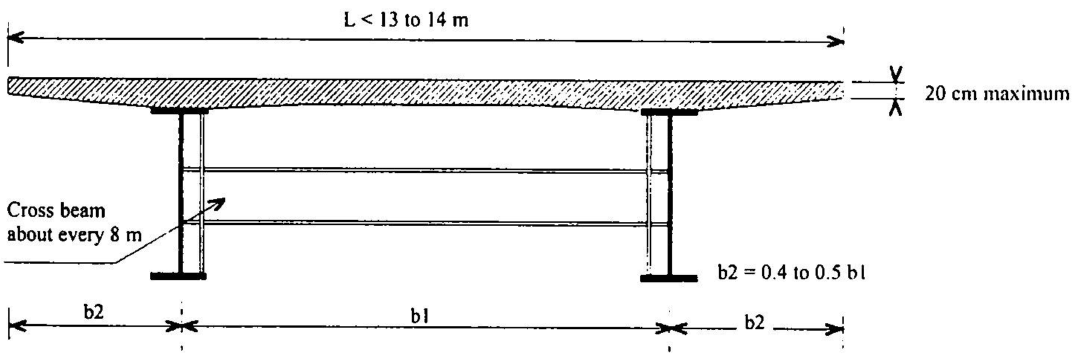

Fig. 1 Composite by-girder structure with cross beams

The traditional bridge (fig. 1) has two longitudinal girders, bounded to the concrete deck slab by shear connectors. A limited number of cross beams, welded to the vertical stiffeners, binds the main girders. This type of bridge can be used both for road and railway traffic and allows spans over 100 m.

For wide bridges, the present tendency is to keep as much as possible the by-girder system, either by transversal prestressing in the deck slab or by the use of more cross beams closer to each together and a cantilever external slab.

By-girder bridges with a lower deck can also be used for both road and railway traffic as this High-Speed Rail bridge in France or this magnificent small ROMANCHE bridge due to the engineer Tonello.

For spans no longer than 60 m, a even simpler design than the by-girder system is made of two small rectangular box-girders with no stiffener, no cross beam, no transverse member, but only a diaphragm at the bearings on pile. This solution, due to its simplicity and its small surface to be painted, is very economical to built and will also have low maintenance costs.

The comparison of the lengths of the lines of welding, the transversal butt joints and the surfaces to be painted easily shows the interest of this design. It is obvious that the most recent methods to check the stability of the plates have to be used to come up to such a simplicity. Some national regulations, as the Swiss and Belgian standards, allow the evolution of the calculation methods. Other ones, more conservative, do not allow it yet. The Bridges Eurocodes are indeed still to come.

As much as possible, the box girders with stiffened bottom are avoided for they are less economical. Their high torsonial rigidity is an advantage, that makes them necessary as the plane curvature is high or for long spans.

Box-girder bridges allow to cross long spans. In Spain, the nice bridge of Professor Martinez-Calcon, with a 180 m long span, has been a world record. I don't think to be mistaken as I affirm that the CANORI bridge in Venezuela, designed by the Leonard office and later presented by Mr. Saul, holds the present word record with a 213 m long main span. Both these bridges have a bottom of box-girder that is metallic in the zone with positive bending moments and metallic strengthened with concrete in the zone with negative bending moments on bearings.

For many years, researches have been undertaken, on the one hand, by the designers of metallic bridges to get their bridges more economical and, on the other hand, to the concrete supporters to lighten their structures.

Original solutions have come up :
- the CHAROLLE and DOLE bridges, with pleated webs,
- bridges with tridimensional truss as in the Boulogne area in France or in Japan, with a maximal 119 m long span,
- the LULLY bridge in Switzerland, with a triangular truss made of cylindrical tubes.

Truss beams bearing a concrete deck slab constitute an elegant alternative to the classical by-girder system. Requiring higher beams but offering good transparency, these structures ask for exactness and much sobriety in the design of the details and connections.

As examples, the small CRUCHTEN bridge, built with higher elastic limit Histar steel and the new BLOIS bridge, with a truss of variable height, for which the lightness of the structure has unfortunately to suffer from a lack of delicacy in the design of the bearings.

The most beautiful and impressive examples of composite truss bridges are railway bridges. The most famous one is the NAUTENBACH bridge in Germany, with a 208 m long span. Others examples exist too in Austria. All these bridges will be presented during this session.

Another bridge, with shorter spans but very elegant, is the ARC bridge in France, presently under construction. The great Spanish engineer Torroja has already designed quite a few of these bridges...

Another particular truss bridge is the DREIROSEN bridge in Switzerland, with two roadway levels. This idea is more and more largely used, even for very long cable-stayed bridges.

4.2 Arch bridges

A transition can be made from the truss girder bridges to the arch bridges with two examples of original bridges :

- the ANTRENAS bridge in France, half arch, half spatial tubular truss,
- another bridge in Czech Republic, with a metallic tubular arch filled with concrete.

During these last two decades, the long arch bridges have been built in concrete, which seems quite normal for an mainly compressed structure. However, a few particular bridges with interesting characteristics show the composite structure as the solution of the future.

In Italy, a small steel-concrete composite arch bridge shows how easy can be the construction of this type of structure.

On a flattened concrete arch, to limit the stresses in the concrete, a light composite deck is launched. That is the ROCHE BERNARD bridge in France.

In Spain, several metallic arch bridges with a composite deck are really beautiful. The RICOBAYO bridge has to be pointed out.

In Belgium, an arch bridge with 270 m long span (fig. 2) has a particularly small height for the arch elements (1/100 of the span); this gives a very light and transparent structure. The use of thick plates and the small cross-section of the box-girders allow to avoid stiffeners and wind-bracing and, therefore, to realize an economical bridge.

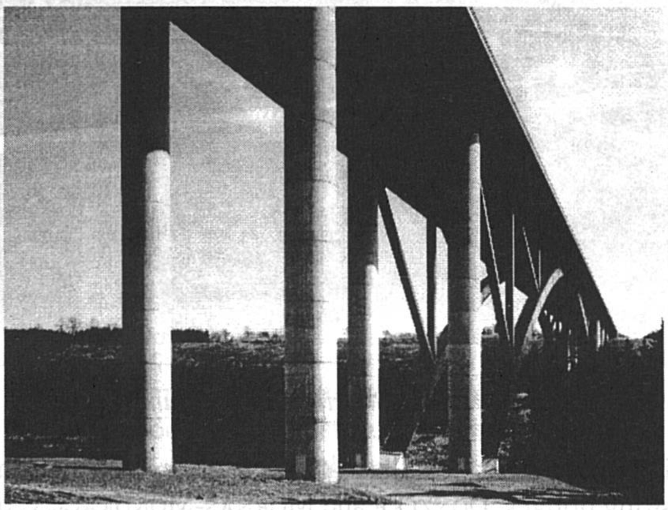

Fig. 2 EAU ROUGE viaduct in Belgium

We firmly believe that this type of arch, much lighter and less sensible to the quality of the foundation soil and to the seismic effects, has a real future towards similar concrete bridges.

The most marvelous examples come from China, which is the topic of another paper. During these last years, the Chinese engineers have built several arch bridges whose metallic tubes were filled with concrete. The longest one, word record with a 420 m long span, is the WANXIAN YANGTZE RIVER bridge, presently under construction. In this last case, the metallic tubular arched is coated with concrete.

4.3 Bowstring bridges

These bridges mainly comprise a compressed chord, the arch, and a tense chord, the deck. They allow numerous variants contravening to the base principles of the composite structures. Most often, the arch is metallic and the deck is either in concrete or steel-concrete composite.

An example of this kind of bridge with a prestressed concrete slab, the CHANXHE bridge, is under construction in Belgium. Another one is the RONQUOZ bridge in Switzerland.

A concrete arch showed unacceptable fissures, the prestressing of the tense deck was insufficient and the bearing outfits were too weak. The replacement of the concrete arch by a much lighter metallic arch was the solution to all these problems.

Bowstring bridges, with an approximately 150 m long span an a steel-concrete composite deck are very elegant solutions to cross a river or a canal, as the HERMALLE (fig. 3) and MILSAUCY bridges in Belgium.

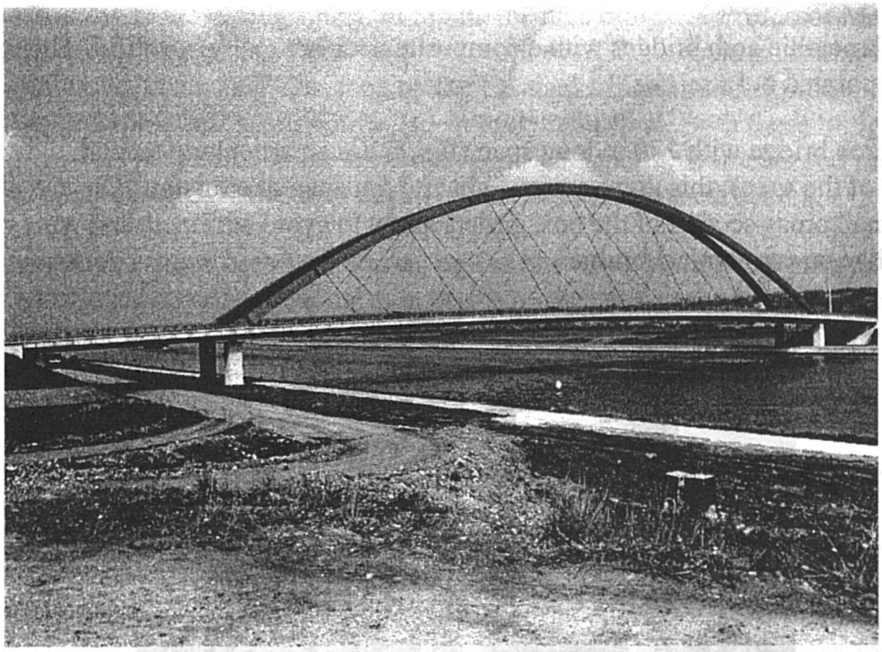

Fig. 3 HERMALLE bridge in Belgium

On the Mediterranean High Speed Rail track, the National French Railway Company is presently building composite bowstring bridges, DONZERE and MORNAS-MONDRAGON, particularly well adapted to the fatigue resistance and dynamic behavior requirements.

Once again, we have to point out the Chinese bridges with cylindrical tubular arches filled with concrete et spans up to 200 m.

4.4 Cable-stayed bridges

The cable-stayed bridges are without any doubt the most fashionable modern bridges at the present time. Here too, the composite structures are interesting, both in the overall design and to solve particular problems.

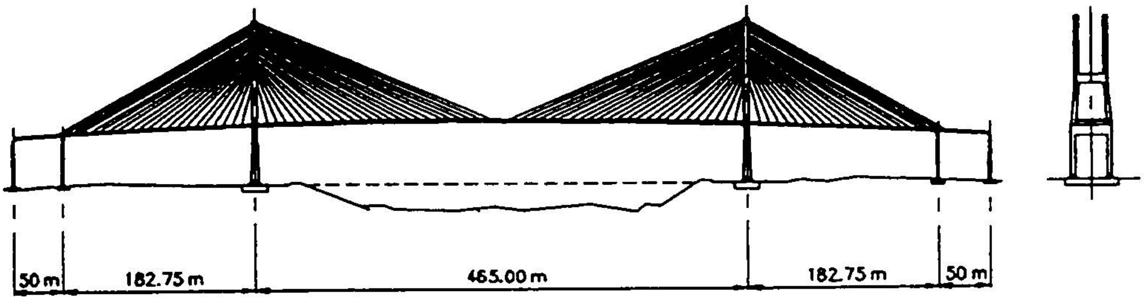

Fig.4 ANNACIS cable-stayed bridge in Canada

Perhaps is it useful to remind of the ANNACIS bridge in Canada (fig. 4), composite cable-stayed bridge with a 467 m span, absolute word record for cable-stayed bridges for many years. Presently, the YANGPU bridge in Shanghai (fig. 5) has a 603 m long span with a similar design, two longitudinal beams supported by two layers of stays et bounded together by cross beams bearing a reinforced concrete slab. The NORMANDY bridge, present world record for all categories cable-stayed bridges, is also a composite structure, as its central span is metallic and the balancing spans are made of prestressed concrete.

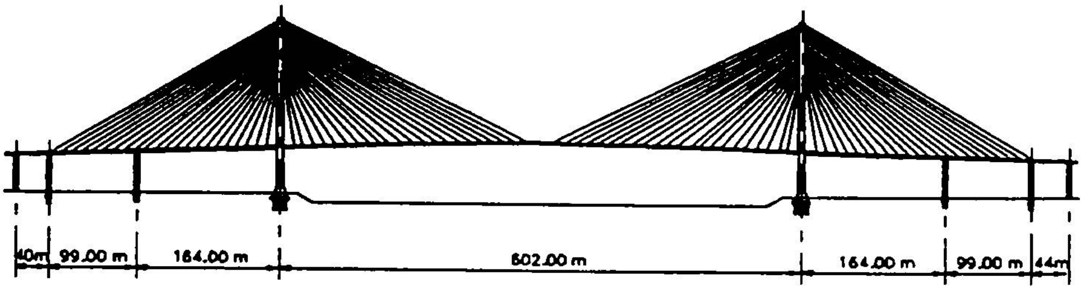

Fig.5 YANGPU cable-stayed bridge in Shangai

The metallic cable-stayed bridges, less expensive than the suspended ones, are now believed to reach 1500 m spans between pylons. The experience shows that the full metal deck with orthotropic slab is mainly dedicated to the very long spans. Up to 700 m or maybe even 800 m, the composite structures can be reasonably considered as the optimal solution.

The bridges built these last few years show the trend of those structures. The ANNACIS and SHANGAI bridges are typical examples : the classical "composite cable-stayed bridges". This denomination is usually understood as a cable-stayed structure with metallic beams and a concrete deck slab, as pointed out earlier. The objective is of course to use at best the best characteristics of each material.

The steel allow the prefabrication in workshop of small beam elements, quite light and very resistant, with precise dimensions and easy to assemble. This prefabrication occurs as the foundations and the pylons of the structure are built on site, which reduces the construction time towards other techniques.

On the other hand, those classical composite bridges rely, more the other full metal or full concrete decks, on the construction method.

Indeed, for a classical composite section, in a very simplified schema of longitudinal behavior, the shear forces and the bending moments are handled by the metallic beam and the concrete deck slab takes the big compressive forces due to the inclination of the stays. That is why the composite bridges built by the cantilever method are designed following the by-girder type with two layers of hangers.

In that case of two layers of hangers, the stays can be fixed either directly on the longitudinal beams, where the balance of the horizontal and vertical forces is realized, as for the ANNACIS and YANGPU bridges or on very rigid transversal beams that transmit the forces of the stays to the main beams, as for the new SEVERN bridge in England.

The construction of the deck on site has to be scheduled very precisely. The metallic beams are placed in balanced cantilever. For each beam element, a hanger has to be placed to bear it. The transversal elements, the cross beams, complete the metallic structure.

The construction of the deck slab, either in precast elements or concreted on site, will follow regularly the assembly of the metal and actually, as close as possible. Let us recall that the concrete slab handles the major part of the compressive forces in the deck. The slab has therefore imperiously to be efficiently connected as early as possible to the main beams, which, as said earlier, are bounded to the stays.

The most delicate things to be treated in this kind of structure are the following ones :

- the steel-concrete connection, of course,
- the anchorage of the stays and the transmission of the vertical forces to the deck slab,
- the effects of shrinkage and creep,
- the problems of wind stability of this composite deck with external girders, whose aerodynamic profile is not really good.

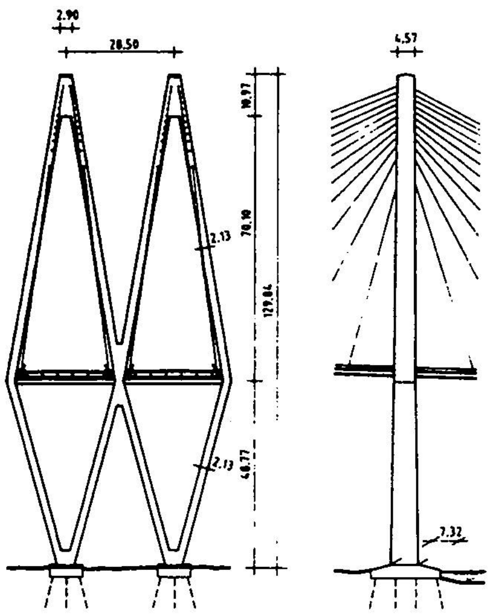

Fig.6 BAYTOWN bridge in the USA - Tower layout

The principles that I just described, the classical transversal cross-section and the construction method, have usually been used to the major existing bridges (listed below), with just a few small differences.

-	YANGPU	China	(1993)	602 m (world record)
-	XUPU	China	(1996)	590 m
-	ANNACIS	Canada	(1986)	465 m
-	HOOGHLY	India	(1993)	457 m
-	SECOND SEVERN	United Kingdom	(1996)	456 m

The BAYTOWN bridge in the United States of America (fig. 6) has the particularity to be constituted of two separate decks and two bounded pylons, which insures an obvious transversal stability.

Three bridges have to be pointed out, as they show noticeable differences towards the classical schema described earlier :

- the RAMA VIII bridge in Bangkok, with a single Y-inverted pylon, two layers of hangers and a wide composite girder-box,
- the KARNALI bridge in Nepal, with a 325 m long main span, a single pylon, two layers of hangers and a composite truss deck,
- the TING KAU bridge in Hong-Kong (fig. 7), with three pylons, two main spans, four layers of hangers, used for the suspension and for the stabilization of the central pylon.

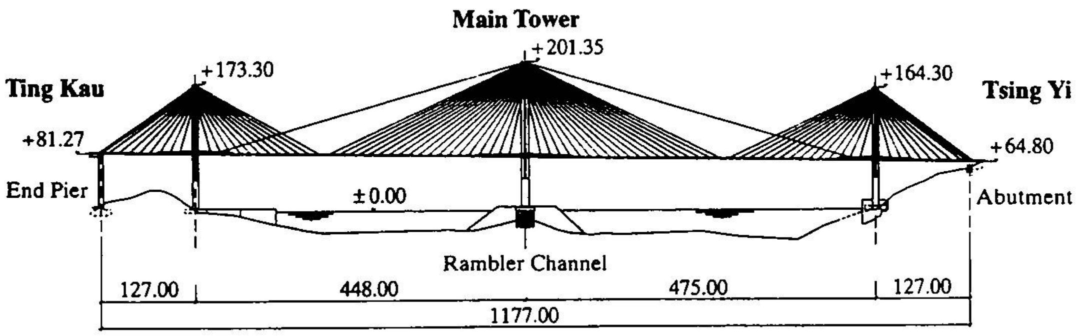

Fig. 7 TING KAU bridge in Hong Kong

The present tendency to use thinner and thinner transversal cross-sections, needing two or even four layers of hangers, comes from the following establishment : the bending solicitations in the deck are proportional to the proper rigidity of the deck if the multiple stays are regularly disposed on the length of the bridge. A direct consequence of the thinning down of the deck is obviously the decrease of the surface exposed to the wind.

However, the aerolastic stability requires a significant torsional rigidity as the span reaches 500 m, even with a double layer of hangers.

If, during these last years, a general tendency has led the designers to thin down the decks thanks to a better knowledge of their behavior, thick decks have to be designed to solve specific problems as for bridges for heavily loaded railway tracks or roadways on two levels. The thin decks are subject to deformations that are incompatible with the railway traffic requirements. That is why truss or high inertia box-girder decks are used.

That is the case for decks with two levels of traffic, as the new KAP SHUI MUN bridge in Hong Kong, the HIGASHI KOBE bridge in Japan and the new ORESUND bridge In Sweden, presently under construction.

The developments towards high performance, as well for steel as for concrete, will soon allow to build composite bridges with a main span probably over 800 m long. Anyway, it will still be necessary to adapt and to improve the aerodynamic characteristics of the decks.

Among those long bridges that impress us by their exceptional dimensions and the achievements that they required, the last ones show some originality. We believe that the composite construction offers a wide diversity of solutions, already pointed out in the last presented bridges. These are smaller bridges, not built with the cantilever method and therefore offering more liberty of design, as for example the SEYSELLE bridge in France, the SAINT-MAURICE bridge in Switzerland and several bridges in Finland.

Another bridge, soon to be built, the KORTRIJK bridge, shows that the combination of two materials can solve difficult problems with much elegance. This small bridge crossing a river in the middle of the town, needed a very thin deck for obvious reasons of navigation clearance and grade profile of the crossing road. Furthermore, for aesthetical reasons and cross-roads congestion problems, a single central layer of hangers was imperative, providing to solve the problem of the balancing stays of the pylon. This has been realized with this stand-shaped metallic pylon, restrained in the prestressed concrete deck.

The ALZETTE bridge in Luxembourg (fig. 8) is a symmetrical cable-stayed structure with a single central pylon and one layer of semi-radial hangers. An important characteristic is the horizontal curvature, rather uncommon for a cable-stayed bridge. The curvature, measured at the longitudinal axis, has a radius of 1750 m.

Fig.8 ALZETTE bridge in Luxembourg

The steel concrete composite deck is composed of two steel trapezoidal box girders with bracing frames and a reinforced concrete deck with a variable thickness. There are no longitudinal stiffener in the webs and in the bottom of the box-girder.

This composite deck, lighter than a concrete box-girder and more economical than an orthotropic slab, forced itself for the transversal solicitations on the pylon, due to the plane curvature, required a light structure.

Another very interesting curved cable-stayed bridge, the ARENA viaduct in Spain, has six pylons and seven cable-stayed spans.

Last example of a particular composite cable-stayed bridge, but it has not been realized. All the movable bridges were, up to not so long ago, metallic structures. However, some movable composite or even totally prestressed concrete bridges are built.

It is absolutely obvious that the extra-weight of a movable bridge has a direct financial impact on the mechanisms and in the electric power consumption. However, for long spans, the wind effects become preponderant in the design of the mechanisms, as well for the swing bridges as for the bascule bridges. That is why the importance of the various parameters of the economical balance of the project evolves and, more and more, the orthotropic slab will also give way to composite solutions with concrete deck slab.

Some interesting suspended bridges with a small main span have also to be pointed out. The deck, either composite or in concrete, combined to the metallic suspension cables, allows to build high quality economical bridges. Two example : the footbridge over the NECKAR river in Germany and the VRANOV LAKE bridge in Czech Republic (fig. 9).

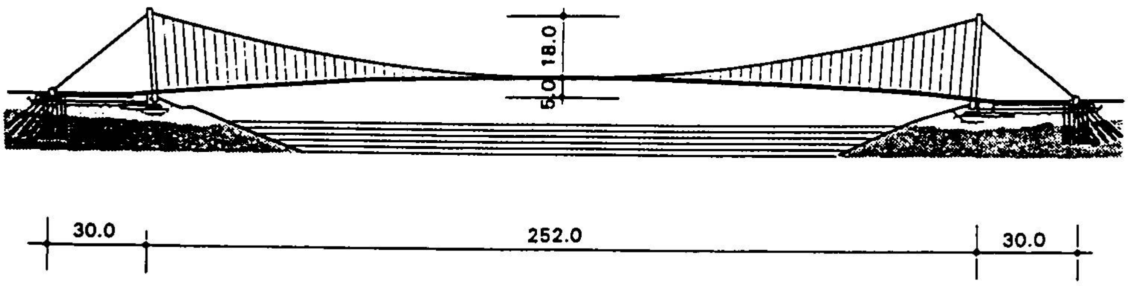

Fig.9 VRANOV LAKE bridge in Czech Republic

5. Conclusion

The impressive quantity and the variety of composite bridges having been built during the past two decades show enough how this kind of bridge has fine prospects before it.

With the increasing length of the spans, the steel continues to be the indispensable material, as the composite structures, in spite of some reticences, compete more and more the prestressed concrete bridges for medium and smaller spans.

These composite structures will be all the more competitive as the regulations will allow us to design simple structures, limiting or avoiding the expensive stiffeners.

We cannot forget that the composite construction requires from the engineer a good knowledge of both materials, without apriorism but with the constant care to question established ideas.

If, in addition, the engineer disposes of performing calculation means and if the regulations, too often retrograde, allow him to reap advantage of them, he will be able to design economical, efficient and high quality bridges.

The harmony of a composite construction is expressed at best with this footbridge in Japan, the INACHUS bridge (fig. 10).

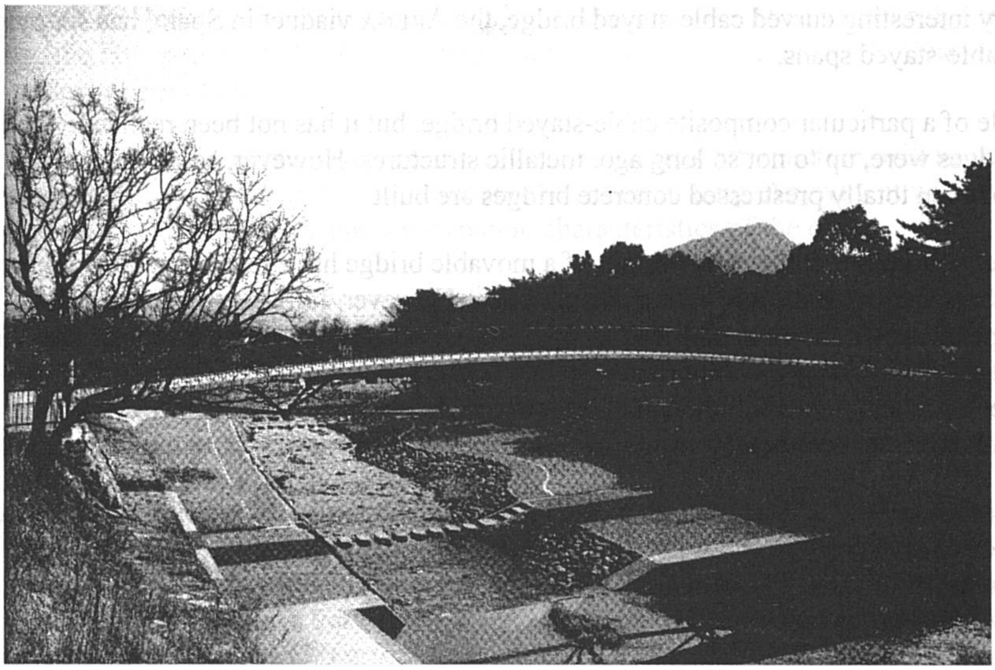

Fig.10 INACHUS footbridge in Japan

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Composite Bridges in Austria

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Summary

This paper describes the situation of composite bridge construction in Austria illustrated by some representative examples. The replacement of existing structures by composite bridges predominates over newly constructed bridges. In Vienna, Nordbrücke, one of the most frequented bridges of Austria, was successfully upgraded. The work performed is described. On the basis of the dynamic characteristics of the structures, innovative methods of inspection and assessment are applied in Austria, which are also presented.

1. Introduction

Due to its topography, Austria traditionally is a country of bridge builders. The motorway network in the Alps, including the three north-south crossings of Brenner Motorway, Tauern Motorway and Pyhrn Motorway, results in a high percentage of bridges in the overall road network. Since the beginnings of reinforced-concrete bridge construction, it was tried to combine steel and concrete in a way to ensure that both materials fulfil the functions best suited to their properties in a bridge structure. This means that the compressive strength of concrete as well as the tensile strength and compressive strength of steel are used specifically, while their interaction is safeguarded by efficient and permanent doweling. In spite of the clear technical advantages of composite steel bridges, this construction method was seldom applied in Austria, as heavy competition – similar to the conditions in our neighbouring countries – made co-operation between steel and concrete construction companies difficult.

The time has not yet come when composite bridges offer advantages in economic terms. However, they meet today's higher quality standards in modern bridge construction and succeed over other construction methods, in particular, in case of difficult basic conditions. Composite construction frequently offers absolutely new design options, such as lower construction depths or low dead weights for medium-span bridges, and often allows for good solutions under difficult external conditions. In this field, composite construction can absolutely compete with prestressed concrete construction, while all-steel construction is only advantageous under extreme basic conditions in Austria. Based on the development on the European market which shows a strong increase in steel bridges, the construction of composite bridges is expected to experience a revival in Austria, too. In our view, the direct benefits of this method are as follows:

- Composite construction is a simple bridge construction method which does not require heavy temporary scaffolding during building so that the terrain below the bridge need not be used.
- Another advantage of mounting is that it takes very little time and, thus, projects can be completed earlier.
- Composite structures can be modified easily which has frequently resulted in substantial advantages when bridges had to be reinforced or widened.
- Concrete decks damaged in most cases due to the action of de-icing salt can be easily repaired, removed or replaced. Likewise, advantages are found when analysing the costs of composite bridges throughout their life. In particular, steel recycling will contribute to covering demolition costs.
- Demolition work is less difficult than in case of prestressed concrete structures.

Due to intensified competition resulting from the economic situation in Europe, steel construction has to attempt to maintain its competitiveness by cutting costs. In this context, potentials are opened up by the following:

- The advantages of composite girders, mainly as regards shear strength, have to be fully utilised. Attention, however, has to be paid to carefully detailing the connections in order to prevent damage which already occurred in the past.
- A more economical design of the reinforced web of main girders seems to be possible. By slightly raising the amount of material used, it is possible to reduce high labour costs.
- An absolutely essential advantage results from the application of concrete decks in wide-span composite girder bridges. The simple principle of using the best suited material at the right place can still be refined, as is shown by some examples.
- The utilisation of even more far-reaching innovations, such as detachable bonds, timed shifting in mounting and the use of prestressed concrete components, let us expect numerous innovative solutions for composite bridges.

2. Composite Bridges in Austria

The great majority of our composite bridges were built during the extension of the high-ranking road network in the 1960's and 1970's. This development started here, near Innsbruck, when the Inntal and Brenner Motorway was extended, and finally spread to the east in the course of the construction of other Alpine crossings and of bridges across the River Danube. Thus, we will start our presentation of examples in the west, near Innsbruck, moving east towards Vienna. Along the Brenner Motorway, there are numerous examples for the successful application of composite bridges. In the course of rehabilitation works carried out after thirty years of extreme utilisation, composite structures are executed again and again. For example, Steinbruch Bridge, a prestressed concrete bridge with a clear span of 5 x 20 m located at a hillside slope involving the risk of slides, was replaced by a composite bridge with a clear span of 100 m.

Miezener Bridge was widened without the main girders having to be reinforced. Just the wind bracings and one bridge bearing had to be reinforced. The lanes of Gschnitztal Bridge were widened in the years from 1986 to 1988 since operation demanded an additional lay-by for safety reasons. As a result, the old reinforced-concrete deck was removed and replaced by a new, thinner deck with limited transverse prestressing.

In the first place, the higher traffic load which results from the widening of the bridge lane would normally lead to an overload for the steel load-bearing system. Given the simplified calculation methods applied at that time, which were on the safe side without exception, and given the higher traffic loads in relation to the bridge width, which were stipulated at that time, it was possible to prove with today's sophisticated calculation methods that the bearing capacity is safe for the reduced traffic load permitted in relation to the increased bridge width without having to reinforce the steel load-bearing system. As these high valley bridges did not have any inspection wagons, such wagons were installed in the two superstructures before widening was started. They were screwed to each of the outer main girders by means of one vertical rail and two horizontal rails using brackets. The vehicles were designed in such a way that it was possible to use them also as mounting aids. The removal and the replacement of the deck was performed from this mounting wagon with the aid of a temporarily mounted rail.

The entire deck was removed and finally produced new section by section. From the block dowels, the loops were removed and replaced by headed shear connectors. The new reinforced-concrete deck was fully doweled also in the support zones. At the steel superstructure, only the lower wind bracings had to be reinforced by welding on cantilever segments. In order to improve fatigue strength, reinforcing butt straps were installed. Transverse prestressing of the concrete slab was performed after completion of steel construction work. In parallel to widening the bridge, its bearings and expansion joints were rehabilitated. Bearings maladjusted in construction were corrected, thus resolving the problems observed in the behaviour of the supporting structure. The examples described show that the rehabilitation and adaptation of all-steel bridges is the most cost-efficient method by far. The costs of rehabilitation amounted to an average of US $ 850 per square meter.

In the second half of the 1960's, „Pilzbrücken" – slim concrete structures with joints in every span – with a total of approx. 56,000 m² were constructed in the motorway section Innsbruck – Brenner. The supporting structure of this bridge type is produced by combining span-sized, single-support structures. „Halbpilzbrücken" with a span width of 15 m were constructed for the lane facing the valley and „Vollpilzbrücken" with a span width of 30 m for the entire cross-section of the motorway. Due to leakage in the numerous system-inherent pin joints and due to the action of the de-icing salt, corrosion heavily damaged the reinforcement and the prestressing anchors which required thorough rehabilitation. Brenner Autobahn AG invited several consulting engineers to participate in a design competition which had the objective of developing suitable rehabilitation plans taking into account the constraint that traffic could be restricted to one lane per direction, but never interrupted or detoured and that 2 x 2 lanes had to be available during the summer holidays.

The plan for the rehabilitation of Reichenbichl Bridge specified the complete renewal of the supporting structure since this seemed to be the only possibility for eliminating all the defects completely and permanently. This rehabilitation plan is based on two innovative ideas:

- Without affecting traffic, a prestressed crosshead is joined to the existing hollow stanchion of the old structure. The crosshead essentially transmits its load via friction, supported by circumferential prestressing, to the stanchion and does not need any other connecting elements.

- For the „Vollpilzbrücken“, a phased removal plan was developed which was co-ordinated with the construction stages of the new composite structure. As a result, both the remaining cross-section of the old supporting structure and first parts of the new structure can take over traffic functions offering an absolutely safe bearing capacity.

This first rehabilitation project was performed on schedule from autumn 1992 to June 1995. As the implementation of the plan was successful in all respects, the client decided to have other „Pilzbrücken“ rehabilitated also in accordance with this concept. At present, the design plans are implemented at Große Larchwiesen Bridge and Weber Bridge, and will also be applied to the rest of the „Pilzbrücken“.

As part of the Tauern Motorway (Salzburg – Villach), Gasthofalm Bridge is located directly before the entrance to Tauern Tunnel.

Effective spans: 52.85 + 5 x 66.06 + 52.85 = 436.00 m
Bridge width: 16.25 + 13.75 = 30.00 m

The supporting structure is a composite bridge with an S-shaped layout. For each direction, a separate composite steel structure was erected. The bigger width of the lane to Salzburg results from the inclusion of a climbing lane. The two plain main girders with parallel chords per supporting structure have a construction depth of approx. 3.50 m and are continuously curved. Due to the layout of rhomb-shaped, horizontal web systems, two parallel torsion boxes are created. The composite deck made of B 400 steel has a standard thickness of 25 cm and is haunched to 40 cm at the main girders. The separate supporting structures of each direction were mounted by cantilevering in parallel and at the same time. The composite deck was erected using a formwork transport wagon with concrete being placed in a staggered way. In the support zones, concrete was placed after the span areas were finished in order to prevent tensile stress in the composite deck due to the concreting load. The total weight of the steel structure, which is mainly made up of special Alfort steel, amounts to approx. 2,000 ton, corresponding to approx. 150 kp/m².

The Altersberg hillside bridge is also located at Tauern Motorway. This bridge was designed and erected in the years 1974 to 1975. Its spans are 48 + 6 x 78 + 2 x 82 + 90 + 69 = 839 m, and its total width is 25.5 m. There is one common structure consisting of four main girders carrying a single concrete deck. The plan of the bridge shows a curvature with radii between 2,500 m and straight line. The main steel structure consists of welded plate girders (height 3,55 m) with distances of 5.5 + 7.3 + 5.5 m, partially connected with bracings and lattice cross girders. The steel qualities are St 37 T, St 44 T and ALFORT (permissible stress 288 N/mm²). The concrete deck is of quality B 400 without prestressing. To reduce the tension stresses at hogging moments a longitudinal compression force (up to 40,000 kN) was produced by hydraulic jacks installed in gaps across the deck. To enable this the concrete plate had a slide way on the upper flange of the main girders. The erection was executed step by step from one side with a derrick crane which set in 18 m long parts of the main girder and after this was moved forward to take over the next parts. Before reaching the following pier a special pantented cantilevering „bill“ was used to take over the bearing force by lifting the end of the bridge with hydraulic jacks. This action reduced the cantilever moment at the previous pier.

The Niederranna Bridge across the River Danube is located at Ebenhoch Provincial Road near Wesenufer, approximately half-way between Linz and Passau.

Effective spans: 91 + 137 + 91 = 319 m
Total width: 13.5 m

The supporting structure is a three-span composite steel bridge with continuous bond. The haunched, plain main girders have a construction depth of 3.75 m in the slab area and of 6.00 m in the support zone. The reinforced-concrete deck made of B 400 is doweled to the top chords of the main girder along the entire length of the bridge and is not prestressed in the longitudinal direction. A state-of-the-art crack-control reinforcement system prevents the formation of harmful cracks. The standard thickness of the composite deck is 25 cm which is raised to 40 cm near the main girders.

Special mention has to be made of the mounting technique. In parallel to the construction of the substructures and the piers, the prefabricated steel construction was assembled into five large elements at the right bank. These structural elements which were up to 95 m long and weighed up to 300 ton were placed by floating cranes within four days. This method did not require any mounting supports in the Danube and resulted in a substantial reduction of the construction period. In order to minimise tensile stress in the composite deck, an optimised concreting sequence was selected. The basic principle was to place the concrete in the support zones after completing the span areas. To reduce tensile stress in the composite deck even further, it was raised by approx. 1.30 m at the abutments. The total weight of the supporting steel structure is slightly below 1,000 ton, corresponding to approx. 230 kp/m².

The Steyregger Bridge across the River Danube is located at the federal road B3 and provides the area to the east of Linz with a direct connection to the industrial zone of Linz.

Effective spans: 3 x 80.6 + 161.2 + 50.6 = 453.6 m
Total width: 24.86 m

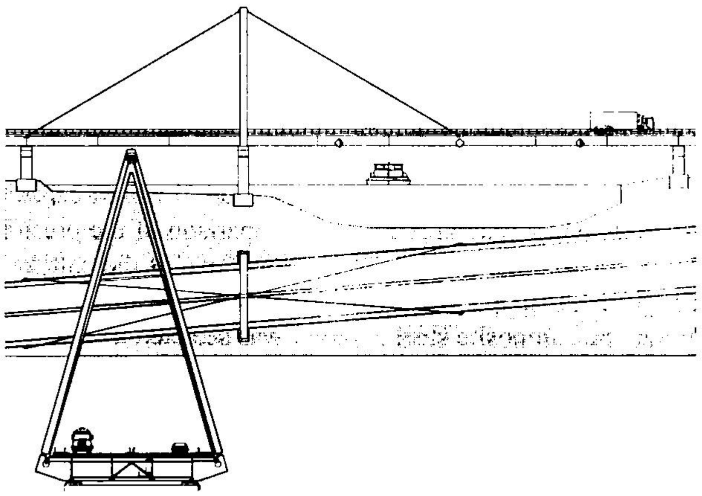


Figure 1: Steyregger Bridge across the River Danube

Since the objective was to achieve a carriageway gradient as low as possible above the clearance defined for navigation, the most economical solution was to construct a cable-stayed bridge taking into account the main navigation channel's width of 161.2 m. Comparative calculations demonstrated that composite construction was more economical than a steel deck solution. Therefore, a composite cable-stayed bridge was constructed. The bridge beam is guyed via an A-shaped pylon in the main navigation channel. The steel structure of the longitudinal girder is a continuous girder grille with four main girders and one load-distributing cross girder per span.

The reinforced-concrete deck made of B 500 is doweled to the main girder's top chords along the entire length of the bridge and pretensioned in the longitudinal direction. The deck has a thickness of 20 cm at a main girder distance of 6.4 m and is haunched to 35 cm above the main girders. The upper bracing between the inner main girders (which was placed slightly lower due to the formwork wagon) absorbs wind load before the deck hardens. In the cable installation zones, the bracing was widened to the outer main girders and allows for cable force distribution to all four main girders. The A-shaped pylon (approx. 44 m high) is based upon the cantilever arms of the pylon cross girder via pivoting point bearings. In the pylon head, the cables run across a welded saddle bearing. Each of the two cable trains is made up of 15 locked-wire strand cables, Ø 69 mm, with multi-layered round core and three Z wire bearings the outermost of which is hot-dipped galvanised. They were pre-stretched by the manufacturer.

The mounting of the bridge was started at the Steyregg end span using two temporary frames and was continued to the Linz abutment by cantilevering. In the second and third span, only the inner main girders were cantilevered to the pier from the middle of the span on while the outer ones were constructed after fixing the inner main girders. After the pylon was erected, cantilevering continued in the navigation channel. Due to the installation of the cable train, the clearance was bridged without interruption of navigation. The deck was concreted in sections with a length of approx. 40 m using a formwork wagon travelling in the top wind bracing so that tensile stress in the deck was minimised. By raising the bridge's ends, in particular the support zones 1 and 4 were relieved. Apart from longitudinal pretensioning by means of tendons (strand cable St 160/180) with a total force of 9,000 ton in the pillar zones, the deck was prestressed via the diagonal cables by raising the saddle bearing at the pylon. Due to the optimised concreting and prestressing sequence, the bonding effect was ensured along the entire bridge. The total weight of the steel construction, including the pylon, cables and installations, amounts to approx. 3,000 ton, corresponding to 260 kp/m^2.

Recently, composite bridges have also been used in railway construction. In the course of the expansion of the western railway line between Vienna and Salzburg, Eisenbahn-Hochleistungsstrecken AG plans to construct a by-pass near Melk in order to raise capacity while providing also a link to the existing train station of Melk. Both the high-capacity section and the by-pass cross the River Melk and a federal road. The comparison of the prestressed concrete solution and composite construction included in the specifications of the call for tenders took into account not only initial investment but also the different costs of maintenance and utilisation. Based on these conditions, the composite steel solution was selected as being more cost-efficient in the final analysis.

The two bridges have haunched plain girders. The high-capacity bridge is a four-span structure (33 m + 48 m + 33 m+ 31 m) having a total length of 146.2 m. The bridge of the by-pass follows the railway track with a radius of 700 m and is a five-span structure (53 m + 53 m + 79 m + 53 m + 36 m) having a total length of 276.2 m. The static calculations were performed according to the new Austrian standards ÖNORM B 4003 and B 4300. As there are no Austrian standards applicable to composite railway bridges, the assumptions were made in accordance with the composite road bridge standard ÖNORM B 4502. A comparative calculation according to Eurocode 1994-2 yielded good correspondence with the assumptions made. The steel grade used was S35510. Since big chords with a thickness of up to 90 mm had to be welded, thermomechanically rolled steel of the grade DIMC-355B was used for improved welding properties. For the conventionally reinforced composite deck, B 400 concrete with a thickness of 40 cm was used for the main bridge and B 500 concrete with a thickness of 50 cm for the access bridge. The steel construction was connected with the concrete deck by means of headed shear connectors. The approx. 1540 ton bridge structure was manufactured in Vienna. The individual components had a total weight of approx. 40 ton, a width of up to 5.2 m and a length of 25 m. They were delivered by special transport to the site and mounted by means of two-engine rubber-mounted cranes.

The Pöchlarn Bridge across the Danube is planned to be constructed in the course of Pöchlarn B 209 road and will link the A1 western motorway to B 3 Danube road. Two lanes and a bicycle path are to be crossed. The total width of the bridge amounts to 13.45 m. In the course of 1997, two parallel call for tenders will be issued in which a composite solution and a prestressed concrete solution will compete directly.

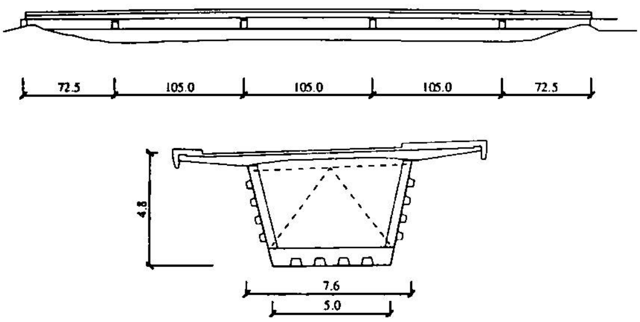


Figure 2: Pöchlarn Bridge across the River Danube

The special feature of this design is the type of erection envisaged. First of all, the steel part of the cross-section will be advanced using the launching method. The concrete slab will be produced analogously by inserting the deck which is concreted section by section at the southern dam and which will only be bonded subsequently. In addition to a slight economic advantage, this construction method is expected to result in significant improvements of the quality of the deck since the fresh concrete will not be subjected to the load which cannot be prevented in conventional construction nor will squeezing due to shrinking occur. Moreover, the deck can be very easily replaced in the future.

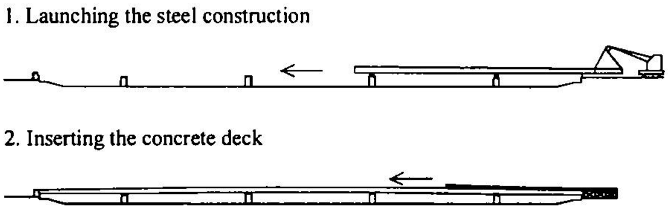
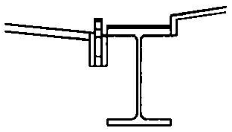
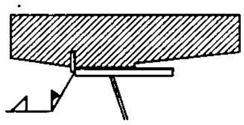


Figure 3: Construction of Pöchlarn Bridge

The bonding effect is produced by a welding seam between the top steel chord and a „Perfobondleiste" which is inserted together with the deck and slightly protrudes over the lower edge of the deck. This protrusion makes it relatively easy to fix the bonding strip in the formwork and also functions as a lateral guide of the deck during insertion. In order to achieve a guide play, each screwed connection of the upper cross arm is fixed for insertion by means of two provisional screws with lower diameters in a slightly displaced position. After insertion, the connections are unscrewed and the top chord is advanced to the bonding strip.



Fixing the Perfobondleiste during concreting



Subsequent production of bonding

Figure 4: Details

In Vienna, there are several innovative composite bridges, such as Kaisermühlen Bridge:

Construction period: 1993 - 1994
Effective spans: 60.3 + 80.6 + 66.95 = 207.85 m
Bridge area: approx. 830 m²

Already during the dredging of Neue Donau, two piers were constructed. At that time it was planned to re-erect the emergency bridges used for the traffic across the River Danube following the collapse of Reichsbrücke as pedestrian bridges at this site. The positions of the piers were adjusted to these 80 m long one-span girders. After this plan was dropped, the position of the piers still was a binding condition for the design of the bridge. The plane environment without any dominance made a significant cable-stayed bridge appear to be a particularly desirable solution. However, the effective span length ratios were very unfavourable for this approach. Instead of guying a specific centre span back to the abutments, the two end spans required an elastic support in this case. As a result, the longitudinal girder of the centre span was stiffened with a triangular truss structure which is harmoniously integrated into the room created by the gradient ascending to the centre. Thus, the non-rigid end spans are elastically stabilised against the stiffened centre of the bridge via bundles of three cables each.

The pylons are rigidly connected to the structure; due to this restraint and the widening of the cross-section in both directions, there was no need for an upper cross arm for stabilisation purposes. The longitudinal girder is formed by two plain girders placed in an inclined arrangement to each other so that the width of the pedestrian level of 4 m visually widens to the top. The footway construction consists of a ribbed concrete slab with trapezoidal sheeting as permanent formwork.

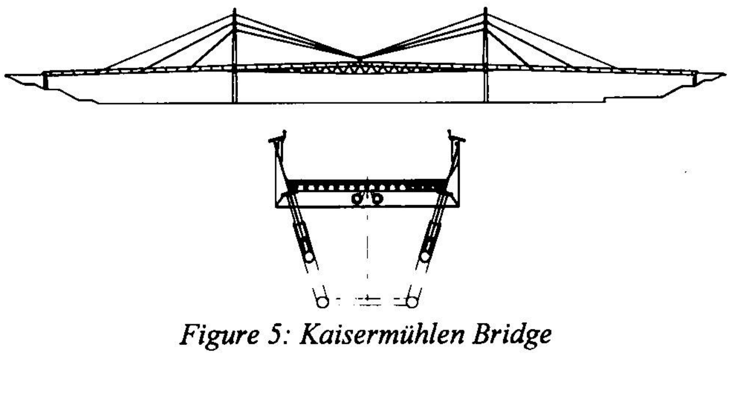


Figure 5: Kaisermühlen Bridge

The supporting sections of the two end spans were welded together along the bridge axis at the mounting sites and launched via temporary frames and barges to the piers in the river. The pylons were already provisionally connected to the supporting structure via joints and launched together with it. The centre piece of the longitudinal girder was mounted to the stiffening truss in parallel to the river bank from where it was lifted by a mobile crane to two barges which transported it exactly below the two cantilevering parts of the end spans. By means of presses and strands, the centre part was lifted to the correct level of the supporting structure where it was welded to the two end spans. Finally, the pylons were folded up, rigidly welded to the longitudinal girders, the cables were partly stayed, the carriageway was concreted and the final cable tension was adjusted.

For the construction of the 20 m wide Ameis Bridge across the western railway line, a one-span structure without piers was to be designed in order to ensure a flexible arrangement of the tracks below.

Construction period:	1982 - 1983
Effective span:	58.5 m
Bridge area:	approx. 1,300 m²

Trumpet-like widenings reaching far into the bridge had to be accepted so that it was not possible to apply a trough-bridge design which had been used in the old structure to be replaced. The already low clearance profile of the railways with 5.50 m above the rail top and the gradient permitted a maximum construction height from 1.65 to 1.75 m at the span's centre as a function of the cross-section of the bridge, which was even reduced by more than 50 cm towards the abutments so that the sickle-shaped girders had a high slenderness ratio of only 1/35. For the one-span structure, the composite solution was particularly well suited since the concrete was to be placed exclusively in the compression zone. This design was clearly more economical than a steel-deck bridge with an orthotropic plate. The 30 cm thick concrete slab, which was required anyway due to the high compressive stress, also offers the advantage of being able to ensure transverse distribution alone due to its stiffness, requiring no further bracings. It was possible to keep the steel part very simple by using nine single I-shaped girders which were only structurally connected by extreme cross girders. The trumpet-shaped widenings are handled by means of two additional, slanted girders.

The concrete strength class used was B 500, as planned, and necessary enhancements were achieved by inserting compressive reinforcements. The increased mass as compared with a pure steel construction improves the vibration behaviour of the one-span structure. The option of installing a vibration absorber was envisaged, but based on subsequent measurements carried out under traffic load it turned out to be unnecessary.

The total hog of the steel girders was 60 cm, out of which 15 cm were attributable to the impact of the concrete's shrinking and creeping. Even though individual deformation components showed slight deviations from the calculated values during construction, the aggregate deformation was still estimated correctly. The main girders received a field connection approximately in the centre of the bridge where it was possible to set up a temporary frame between the tracks. The girder components with a length of up to 32 m were transported to the site by night and placed using a two-engine rubber-mounted crane. The webs were connected by high-strength, friction grip bolts, and the chords were welded on site with the lower chords being made of Alfort steel with a thickness of 90 mm.

3. Nordbrücke

3.1 Rehabilitation of Nordbrücke

Nordbrücke which was completed in 1964 has with 2 x 2 lanes and follows the north-west railway line which was abandoned due to revised plans for Vienna's railway network.

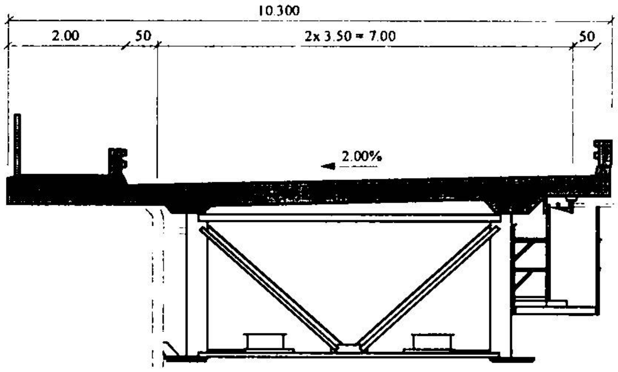


Figure 6: Cross-section of Nordbrücke

This bridge across the Danube was erected in composite steel construction – for the first time in Vienna – with a total of four plain main girders made of steel St 52 with a height of 2.60 m and a span of 83 m. The supporting steel structure was assembled at the right bridge head and launched across the railway facilities and the River Danube towards the left bank. In the years 1984 and 1985, a ramp which already branches off on Nordbrücke was annexed to the composite structure. In this process, the existing cross-section was supplemented by another main girder. Taking into account the creeping of concrete, the achievement of the bonding impact and the connection to the old concrete carriageway deck were timed in such a way that the 30 cm wide connecting joint remained virtually stress-free during concreting.

With its current peak traffic load of up to some 108,000 vehicles per day and, above all, with its maximum load of 2,310 vehicles per lane and hour, this structure is among the bridges with the highest load in Central Europe. After more than 30 years of bearing high traffic loads, the resulting state of the bridge required the performance of the first general rehabilitation. The damage to the deck is largely attributable to de-icing salt whose impact was still unknown at the time when the bridge was constructed. Other systematic damage was caused by the ageing of the bituminous water-proofing and by the omission of the insulation below the central strip and the pedestrian area, which used to be common practice for economical reasons at that time. This damage was monitored and documented in the on-going inspections of MA 29 and, eventually, recorded in a summary assessment which formed the basis for the decision in favour of general repair works.

After the removal of the entire bridge equipment, bituminous surfacing and old water-proofing, the concrete located at the raw structure was repaired and covered by epoxy resin before the new, double-flamed bituminous water-proofing, a levelling course and 2 x 3.5 cm of mastic asphalt were applied.

As pedestrian and bicycle traffic was moved to Nordsteg, the narrow pavements on Nordbrücke were no longer needed. Thus, it was possible to establish one narrow lay-by in addition to the two lanes per direction, without widening the bridge in total, which would have been impossible or at least highly expensive for static reasons. This yielded a considerable increase in traffic safety and a reduced frequency of traffic jams.

For this purpose it was necessary to remove the old footway cantilever and to mount a margin protection in the form of an 80 cm high safety wall at the outer end of the cantilever arm. For the reason of weight alone, the structure was made of steel and consists of steel posts placed at intervals of 4 m to which 8 mm thick wall sheets are screwed. Additionally, a handrail is mounted on them which supports the safety wall in case of accidents as a continuous tieback. The anchorage of the wall posts at the top of the cantilever slab was designed for a static impact force of 100 kN. In order to produce the anchorage, first of all, the outermost 50 cm of the cantilever slabs were removed by means of high-pressure water jets and the reinforcement was uncovered. Additionally, slots were cut, also by means of high-pressure water, for a strong, upper secondary reinforcement concentrated near the posts. At the end of the cantilever arm, an edge beam was concreted which acts as a stiffener and distributes the truck wheel load required by the standards in such a favourable way that it was not necessary to enhance the reinforcement within the standard range. The outer formwork of the edge beam is formed by a continuous margin sheet at the outside of which the posts of the safety wall are mounted.

The mobile scaffolding for concreting the edge beams was conceived in such a way that the loads were introduced close to the main girder. The formwork was pressed to the existing slab by means of anchor bars. On the whole, all the re-structuring measures were designed to have no significant impact on the main system of the bridge. In relation to the overall area of the slab, the changes and additions to the cross-section are of minor importance. The prestressing of the tension zones was not affected in any way.

At its 30th anniversary, Nordbrücke is a practically new structure, furnished with lay-bys, a steel margin protection stiffening the entire concrete deck and new central strips with safety rail damping elements, all that in maintenance-friendly versions. Moreover, a convenient pedestrian link was provided by the Nordsteg replacement bridge. The costs of the general repair works amounted to approx. ATS 200.0 million. Apart from damage due to the action of de-icing salt and moisture penetration, the 30-year old composite deck does not show any cracks and fatigue problems in the bonding area and was turned practically new by the repair works. Since constructional defects were not found in the composite structure either, this bridge is again in a perfect state so that composite construction is recommendable for bridges of this type both in technical and economic terms.

3.2 The Bridge Monitoring System BRIMOS

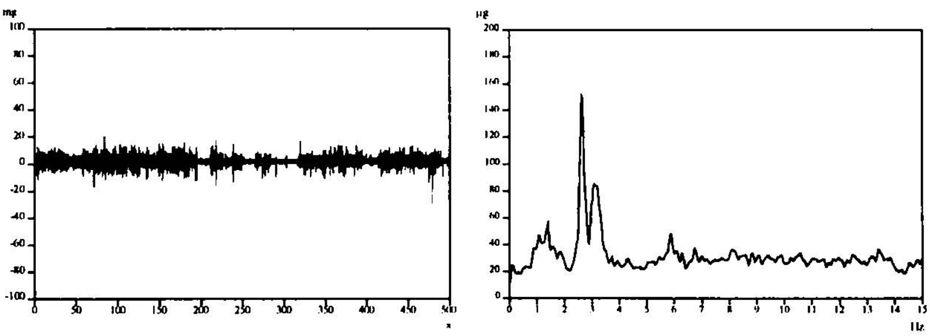


Figure 7 : Typical signal and spectrum of the Nordbrücke in Vienna (composite structure)

For the purpose of „System Identification" a monitoring set-up was created, that enables quick an efficient recording as well as signal processing and report generation. The basis is the measurement of acceleration in a well determined layout of relevant locations of a structure. This provides data for the FFT analysis to generate the desired spectra. In addition data of the actual displacement of the structure is collected by infrared laser to gather information on the static behaviour and its relation to the dynamic action.

3.3 Data processing

The collected data are processed to provide an informative report, which shall contain information on the signals itself in the desired units, the power spectrum of the readings, raw and smoothened, the drift of the readings and the relevant displacements. In a further step the readings of the various locations are combined to get an averaged spectrum and the related displacements. This is the basis for the animation of the modes of the structure and the visualisation of it.

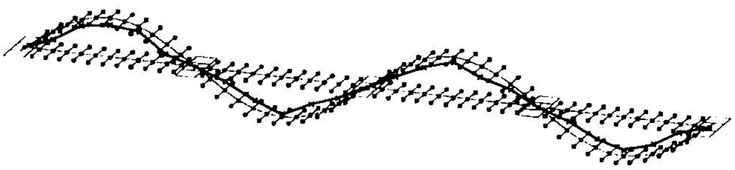


Figure 8: First mode calculated and measured of the Nordbrücke in Vienna

3.4 Conclusion

Due to the fact, that this bridge was monitored during 3 different stages, before, during and after the rehabilitation, valuable information was gained about the influence of the state of the structure on the response spectrum. From this basis it is tried to develop further tools to assess the quality of structures using data from dynamic monitoring.

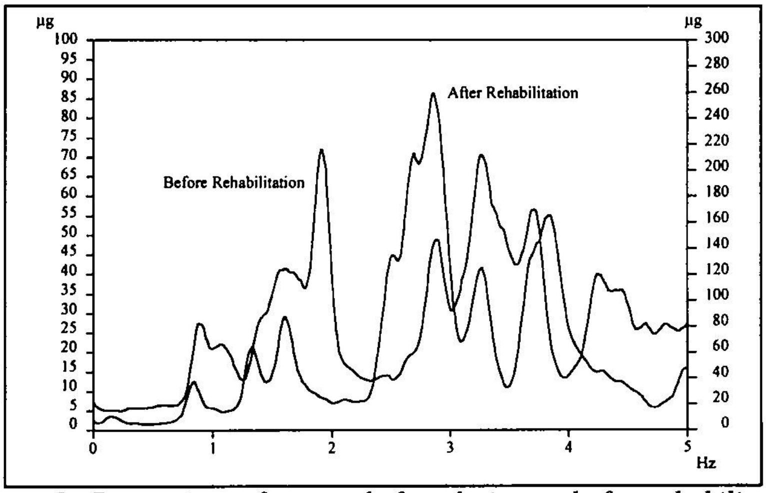


Figure 9: Comparison of spectra before during and after rehabilitaion

In future inspections will be carried out periodically to judge the state of the structure with this system. The calculation of life time cycles will be based on firm data and be more accurate. A benefit in terms of safety and allocation of funds is expected.

Composite Building Structures in Earthquake Engineering

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Federico M. Mazzolani, born 1938, author of more than 350 papers and 12 books in the field of metal structures, seismic design and rehabilitation. Member of many national and international organisations. Presently Chairman of: UNI-CIS/SC3 Steel and Composite Structures; CNR Fire Protection; ECCS-TC13 Seismic Design and CEN-TC 250/SC9 Aluminium Alloy Structures.

Summary

Suitable combinations of constructional materials may generate composite actions which are successfully utilized in seismic resistant building structures. The main behavioural features of such combinations, usually called composite systems, have been examined in case of new building construction as well as in case of old building retrofitting.
After an overview of the international situation both in research and codification, some example of application of composite building structure in seismic areas have been presented.

1. Introduction

The ability of structural typologies to withstand severe actions is particularly proven when the building constructions are submitted to the violence of an earthquake. The examination of damages always represents a precious source of information on the ultimate performance of constructional materials [1].

Referring to the traditional typologies, the old masonry structures are the first to prematurely fail under the seismic attack, due to their intrinsic features which are very often worsened by the age and the revage of time. They need to be upgraded by means of more ductile and modern materials, like concrete and steel, giving rise to different kinds of composite actions.

But unfortunately also many reinforced concrete structures are seriously damaged and sometimes collapse because of the earthquake, due to bad execution and poor material quality, which produce a tremendous lowering of ductility.

Looking to steel buildings the past experience show that the cases of global collapse are very rare, even if the traditional image of steel as the more suitable material in seismic resistant applications has been seriously undermined after the damages recently occurred during the Northridge (17 January 1994) and Kobe (17 January 1995) earthquakes.

Summing up, from the experimental evidence of the sad after-earthquake scenarios, it is easy to recognize that all the common constructional materials from the worst to the best used alone in the traditional typologies can badly perform under severe earthquakes, producing serious damages up to the collapse.

In order to increase the reliability of constructional materials, it can be observed that a rational combination of non-ductile (masonry, concrete) with ductile (timber, steel) materials can produce a kind of synergic effect which improves the behaviour of the construction under severe actions.

Looking back to the historical development of the seismic resistant structures, we can find that, after the catastrophic earthquake which destroyed the Calabria region in South of Italy at the end of 17th century (1783), the government imposed to build the new constructions by using a timber lattice-work inserted into the masonry walls (Fig. 1). The so-called "*casa baraccata*" (*treillis house*) represented the ancestor of a composite structure (masonry plus timber) conceived for seismic resistant purposes and its performance was largely appreciated during the subsequent earthquakes. This system is very similar to the one imposed in Lisbon after the earthquake of 1655.

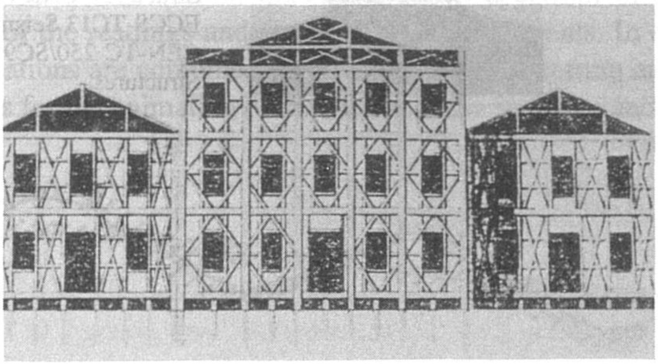


Fig. 1. Timber-masonry composite structure: the first seismic resistant composite system.

Afterwards also the consolidation activity developed in the earthquake prone areas exploited for the first time different kinds of composite actions. We can observe that the composite action has been widely experienced in the retrofitting of masonry buildings, i.e. by means of steel elements for introducing tensile resistance in walls, arches, domes [2, 3] or by using RC plates for transforming masonry walls into sandwich panels.

Only more recently the composite systems have been used for new constructions, but the original motivation was mainly based on economic aspects rather than on the requirement to improve the structural performance. Nevertheless the well known composite structures made of steel elements working together with RC elements demonstrated a good synergic behaviour also under severe seismic actions. Considering the recent damage to connections of a number of steel structures during both recent Northridge (Los Angeles) and Hygoken - Nanbu (Kobe) earthquakes and the numerous failures of new reinforced concrete structures during all the known earthquakes, it appears that the use of steel-concrete composite systems could mitigate some of the vulnerabilities of steel and reinforced concrete structures alone.

2. Main Behavioural Features

In general, the composite action can be defined as an action deriving from the combination of two or more different structural materials acting together to resist external forces. This kind of action can be performed in a single member, in the structure as a whole or in both. On the other hand, it can be derived from the integration of new materials, which are used for increasing the previous resistance of the existing construction, in case the seismic upgrading is requested.

The technological systems allowing the development of a composite action give rise to composite constructions, which can act at two different and separate levels:

A - **member level**: different materials (usually steel and concrete) can form parts of the cross-section; it comprises beams, columns, slabs, walls;

B - **structure level**: sub-systems made of different materials can compose the whole structure; it comprises the possible combinations of frames, bracings, walls and cores, which can be made of simple (steel or RC) as well as composite elements.

Level A leads to the so-called "**composite members**" and level B gives rise to the so-called "**mixed**" or "**hybrid structures**", but all together they belong to the family of composite systems.

From the point of view of seismic resistance, composite systems are suitably used both in the construction of new buildings as well as in the refurbishment of old existing buildings. The main advantages of a composite system in seismic resistant applications, respect to structural steel or reinforced concrete alone, can be identified in the following points:

- high stiffness and strength of beams, columns and moment connections;
- satisfactory performance of all members and the whole system under fire conditions, which can arise after an earthquake;
- high constructability for floor decks, tubolar infilled columns, moment connections;
- increase of ductility for encased beams, encased columns and beam-to-column connections;
- satisfactory damping properties for the whole system.

Due to these synergic properties, it seems logical to utilise the two basic materials (steel and RC) in tandem and to consider, therefore, both composite and mixed structures as an attractive solution to seismic design problems.

Many research results [4, 5, 6, 7, 8, 9, 10, 11, 12, 13] have shown the interest of using composite structures in seismic areas, particularly due to the presence of concrete, which increases the resistance in the elastic field up to 50%, contemporary increasing the stiffness and largely preventing local buckling. After complete concrete crushing, the structure behaves always like a bare steel structure when submitted to very large displacements.

Due to the complexity of the stress state in composite connections many experimental and design-oriented research project have been developed in USA, Japan and Europe for several types of connection details, which demonstrate their potential for use in seismic resistant applications [12, 13, 19, 20]. In terms of seismic design, composite connections often avoid or minimize the use of stiffners comparing to structural steel design.

3. New Building Construction

3.1 Composite Elements

In seismic resistant structures the most commonly used steel-concrete composite elements are: beams, columns, slabs and walls.

In multistory buildings it is very frequent the use of floor structures made of composite beams and composite slabs, which are obtained by casting reinforced concrete on steel trapezoidal sheetings supported by double T beams (Fig. 2). In these cases the main structural advantage of the composite action versus the seismic performance is due to the diaphragm effect which allows to rigidly connect in plan the vertical bracings under horizontal seismic forces.

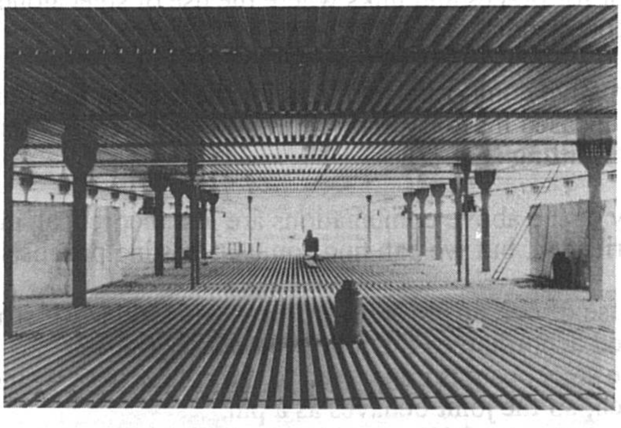


Fig. 2. Steel trapezoidal sheetings before the concrete casting in a composite floor.

Among the composite beams types, the encased beams (Fig. 3) represent a suitable system which provides a good performance under cyclic loading due to the presence of the concrete mass which avoids or at least postpones the local buckling phenomenon in the web of the double T section. This increases the rotation capacity of the member and therefore the ductility of the whole structure. The behaviour of encased beams has been proved by many monotonic and cyclic tests [6, 11].

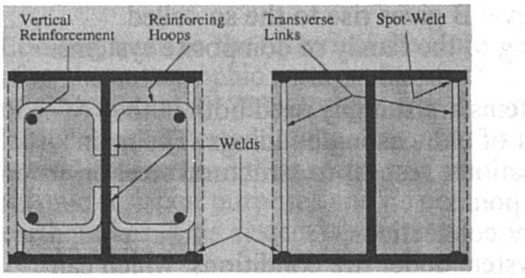


Fig. 3. Partially encased beam-column sections (Elnashai, 1996).

Composite columns made of round, square or rectangular hollow steel sections filled up with concrete present also the advantage to improve the local buckling resistance of the steel wall, allowing to increase the b/t ratio of the section.

Composite walls are derived from the insertion of vertical steel profiles into a reinforced concrete wall. The presence of steel improves the ductibility of the element.

3.2 Composite Sub-Systems

The classification of seismic resistant structures is usually done according to how the bracing system faces the horizontal quakes [14]. The same format can be followed in case of composite structures and, therefore, the main categories of MRF (moment resisting frames), CBF (concentrically braced frames) and EBF (eccentrically braced frames) can be considered.

Referring to MRF composite systems, the following combinations are possible: beams can be simple (steel) or composite (steel plus RC); columns can be simple (steel or RC) or composite (steel plus RC). Figure 4 shows a seismic resistant system composed by RC columns and steel-RC composite beams.

In case of CBF composite systems, beams and braces can be in steel or in composite (i.e. encased sections); columns have the same possibilities as in the previous case of MRF. The EBF leads to the same typologies, eccept for braces and links where the use of steel alone is recommended.

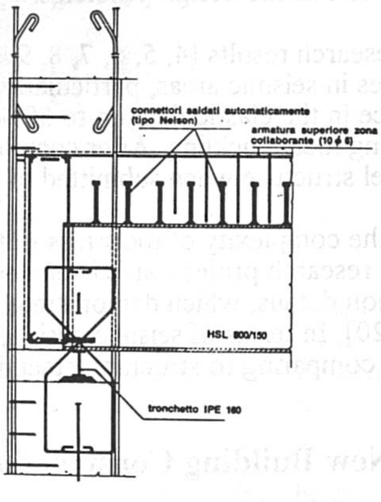


Fig. 4. Composite MRF subsystem made of RC columns and steel-RC composite beams.

Alternative composite solutions for the bracing function can be obtained by means of RC frames with encased masonry and RC frames with steel braces, the last being particularly suitable in case of retrofitting (Fig. 5).

Many of the above combinations are just theoretical; in practice only few cases have been experienced, but we can find some interesting proposals, like the one shown in Fig. 6.

The use of composite steel-RC beams in steel MRF structures provide interesting results when applied according to the following phases (Fig. 7):

- the steel beam is connected to the column tree by means of a bolted cover plate only in the web, so the joint behaves as a pin;
- the concrete slab is casted excluding the part corresponding to the joint, so the dead lead of the floor does not produce bending moments in the columns, except for the affect of eccentricity in the frame node;
- also the beam flanges are connected by means of cover plates, leading to the complete scheme of frame.

The advantage of this constructional procedure (so-called *disconnection technique*) is to provide the structural scheme to absorb negative moments produced by seismic horizontal actions in the nodes by increasing the capability.

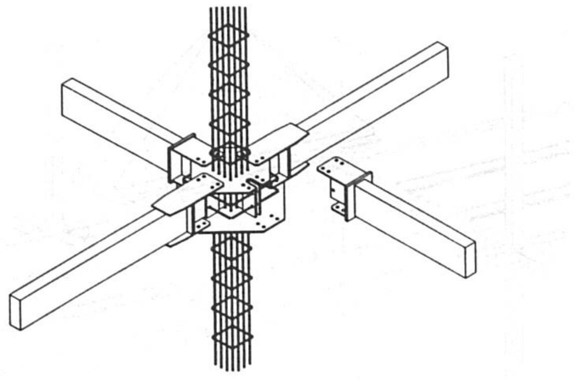


Fig. 6. Composite subsystem composed by RC columns and steel beams for a MRF scheme (Carannante Joints, 1995).

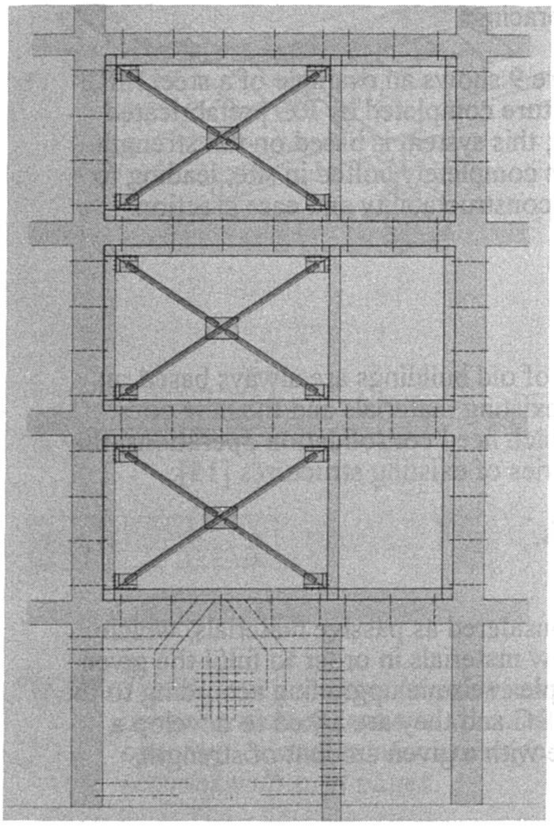


Fig. 5. RC frames with steel bracings.

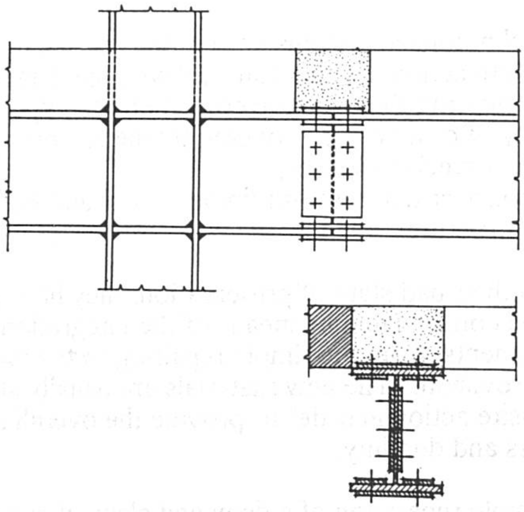


Fig. 7. The disconnection technique of a composite MRF structure for improving the seismic resistance.

3.3 Composite (Mixed or Hybrid) Structures

They can be derived from a combination of different sub-systems which can be simple (RC walls or cores, steel bracings) or composite (like the ones mentioned above in section 3.2).

A very common typology is the one composed by pinned steel frames and reinforced concrete cores and/or walls, in which the two materials play different functions in withstanding the external actions: steel frames provide the carrying capacity of vertical loads and reinforced concrete elements maily resist the horizontal seismic forces (Fig. 8).

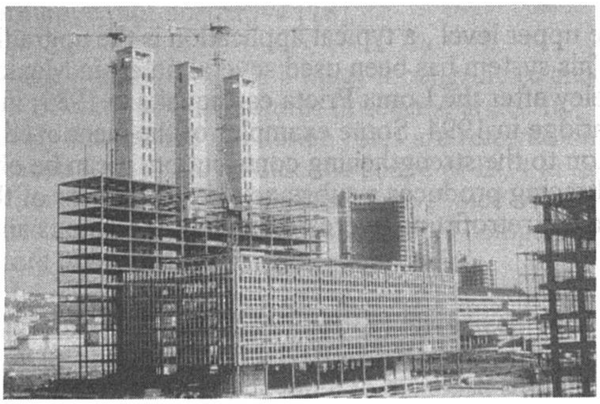


Fig. 8. Composite structure with RC cores and pinned steel frames.

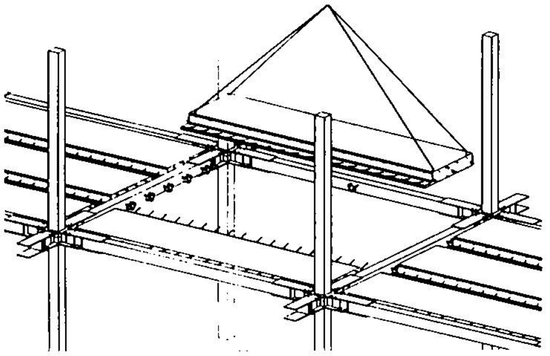


Fig. 9. A composite structure with steel MRF and RC prefabricated slabs.

From the global ductility point of view this solution is not eccellent, because the dissipative zones are concentrated into the RC elements. Nevertheless due to its economical advantages, this type of composite structures is very often used in low seismicity regions. An improvement to this solution can derive from the combination between steel MRF and RC bracings.

Figure 9 shows an example of a steel MRF structure completed by RC prefabricated slabs; this system is based on full strength joints completely bolted in site, leading to high constructability and ease erection.

4. Old Building Retrofitting

The technological systems used in the seismic upgrading of old buildings are always based on composite actions, which can be developped among the existing materials and the new ones. By considering the common constructional typologies which need consolidation operations, the majority of cases can be covered by the following categories of existing structures [15]:

- iron or steel structures;
- masonry structures with timber floors and timber roofs;
- RC structures.

Due to their bad state of preservation, they have to be considered as passive materials, which must be consolidated by means of the integration with new materials in order to fulfil the given requirements, from the simple repairing to the more complex seismic upgrading according to the code provisions. The new materials are usually steel and RC and they are asked to develop a composite action in order to provide the overall structure with a given amount of strength, stiffness and ductility.

The simple reparation of a damaged element can be made in different ways. Some examples are given in Fig. 10, dealing with the main systems using steel as active material for strengthening masonry walls, RC beams, wooden floors [16]. RC can be also used for repairing masonry walls, steel floors, wooden floors.

The main technological systems used in case of retrofitting of existing structures give rise to different composite elements belonging to level A, like: masonry - steel; masonry - RC; RC - steel; timber - steel; timber - RC.

At the upper level , a typical application is the upgrading of RC frames by means of steel braces [3]. This system has been used several times: in Mexico City after the earthquake in 1986; in Berkeley after the Loma Prieta earthquake in 1989; in Santa Monica after the earthquake of Northridge in 1994. Some examples of the mentioned applications are given in the Fig. 11. In addition to the strengthening contribution, it can be observed that in many cases the addition of steel bracing produces a substancial improvement of the aesthetic aspects of the façades, which, before the retrofitting, were completely anonimous and sometimes ugly.

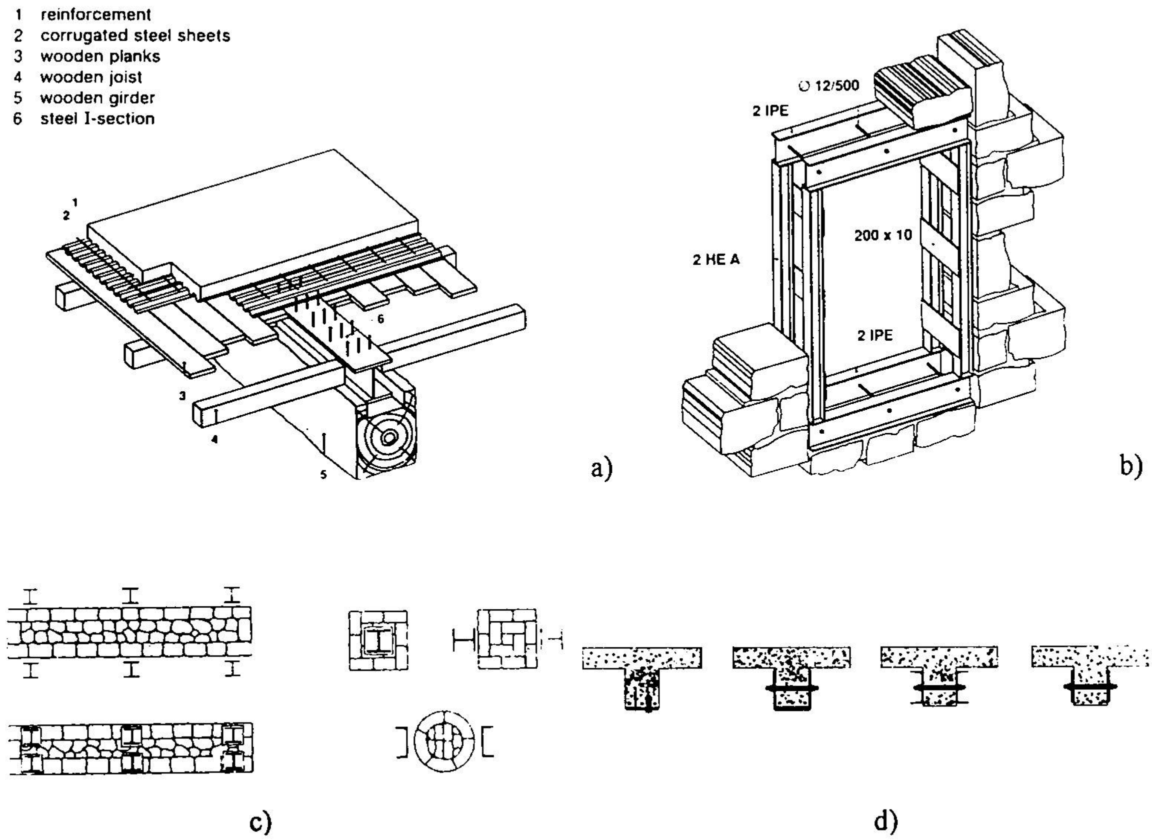


Fig. 10. Technologial systems based on composite actions, which are commonly used in consolidation operations: a) composite steel-timber floor; b) steel frame in a masonry opening; c) steel reinforcement for masonry walls and columns; d) integration of RC sections with steel plates.

5. Codification

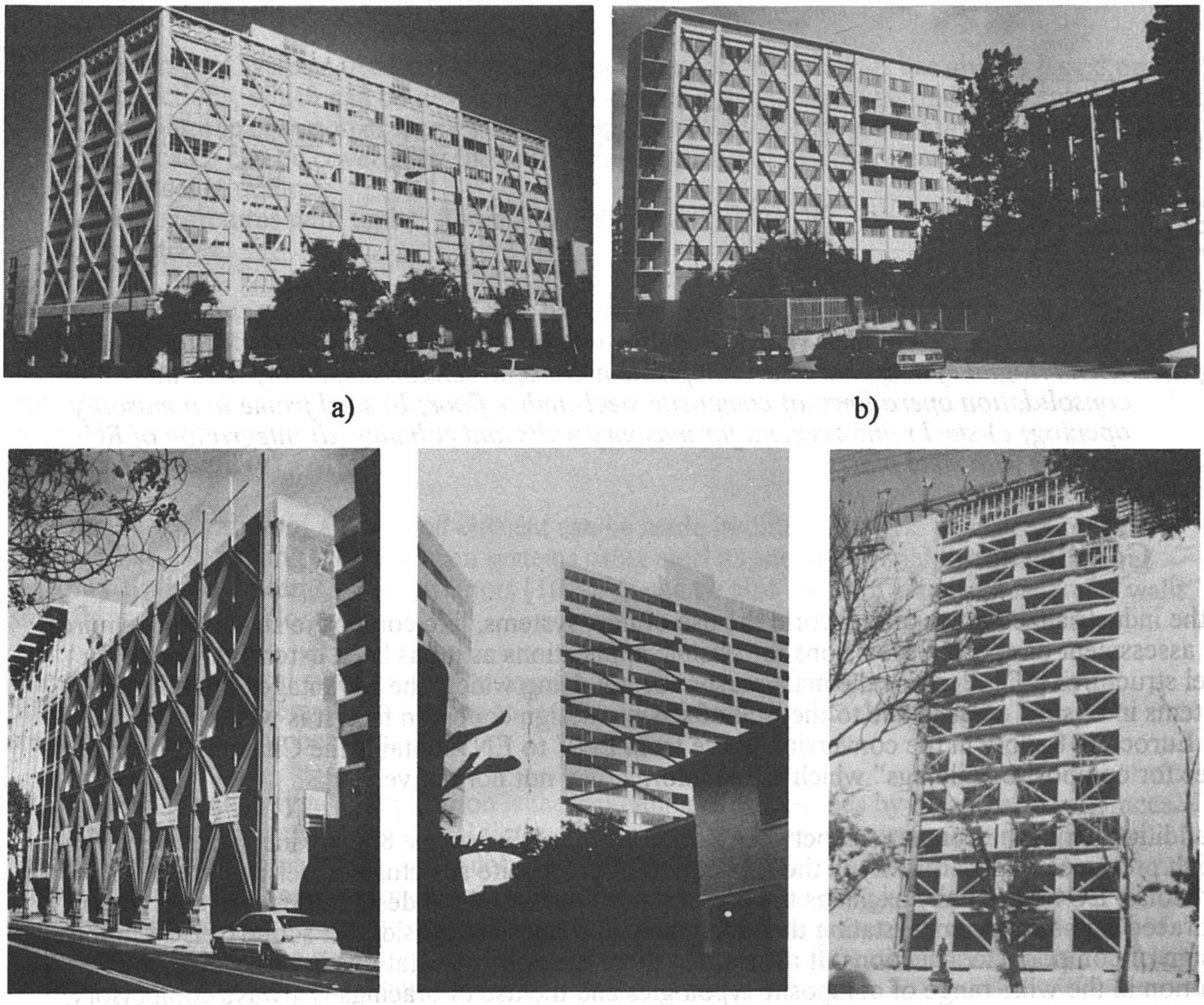
In the industrialized areas of the world the use of new systems, like composite structures, requires the assessment of specific provisions for seismic applications as it has been extensively done for steel structures [17]. Perhaps the main reason for not using widely the advantages of composite systems in seismic areas is due to the lack of seismic design codes. In fact, it is well known that the Eurocode 8, now in the conversion phase from ENV to EN, contains the Chapter "Specific rules for composite buildings" which is just informative, not normative [18].

In addition, from the comparison between Eurocode 4 and Eurocode 8 many incongruicies arise, which produce some perplexity in the application of composite structures in seismic areas. In particular, EC4 esplicitally excludes the use of sway frames and the design rules are referred only to braced non-sway frames, stating that the unbraced frames are outside the scope of EC4 in the design of composite connections. It means that there are strong limitations in the choice of a solution in the wide range of composite typologies and the use of bracings is always compulsory, what vanishes the meaning of the behaviour factors given in EC8 for other typologies.

The first U.S. seismic design provisions for composite constructions (NEHRP) have been developed by the Building Seismic Safety Council (BSSC) within the National Hazard Mitigation Program in 1994 [19]. The most challenging parts of this code are devoted to establish seismic force reduction factors and drift amplification factors. In estabilishing reduction factors, the available research data have been integrated by engineering judgment and phisical understanding of the behaviour of these systems. The basic composite structural framing systems identified in the NEHRP code and the corresponding force reduction factors (R) and drift amplification factors (Cd) are the following:

COMPOSITE SYSTEMS	R	Cd
Special moment frames	8.00	5.50
Ordinary moment frames	4.50	4.00
Partially restrained frames	6.00	5.50
Eccentrically braced frames	8.00	4.00
Special concentrically braced frames	8.00	4.50
Concentrically braced frames	6.00	5.00
RC shear wall with steel elements	8.00	6.50
Shear wall reinforced by steel plates	8.00	6.50

For these systems specific design requirements are provided, with particular reference to connections and detailing.



a) b)

c) d) e)

Fig. 11. Seismic upgrading of RC frames by means of steel bracings: a), b), c) Berkeley; d) Santa Monica - Los Angeles; e) Mexico City.

6. Applications in Building Construction

It is difficult to collect all the existing examples of building with composite structure erected in earthquake prone areas. Some informations have been obtained from the current technical literature.
It seems that in the highly seismic areas of United States the use of composite systems is limited to steel structures with composite floors and more recently to steel structures with concrete-filled composite columns [19]. However, despite the construction of many spectacular high-rise buildings with composite superframes in less seismically active areas, the use of such composite system is going slowly in highly seismic areas such as in California. Nevertheless many research activities have been developed and are now in course in USA [19, 20].

A different situation appears in Japan, where the advantages of composite steel-concrete structural systems are well-documented, thanks to many investigations on different typologies of composite members, connections and frames. In the last twenty years the floor area in square meters of mixed structures rose from 10 to 40 millions about [21].

In Europe an extensive research work has been recently carried out to focus on the cyclic behaviour of composite members and connections [4 to 13]. These studies have confirmed the feasibility of composite frames designed to resist seismic actions, but very few applications for building construction in the European earthquake prone Countries seem to derive from these theoretical basis. As an example, a new steel-composite structure has been recently built in Timisoara - Romania [22], according to the Romanian Seismic Code, which is largely ispired to Eurocode 8.

In Italy the majority of composite (hybrid or mixed) structures have been erected in the area of Naples city and surroundings. Due to the damages produced by the bradyseism phenomenon, the old town of Pozzuoli was completely evacuated in the early eigthies and a new town has been erected for 25.000 people. The pressing need to give hospitality the population in the shortest period of time oriented the choice of the structural typology on prefabricated solutions for low-rise buildings of 4 to 6 stories. The mixed system composed by concrete cores and steel skeleton has been selected in the majority of cases, because of both quick erection and seismic reliability. This can be considered an interesting example of extensive use of composite structures in low seismicity areas.

Fig. 12. General view of the new Management Centre of Naples.

Parallel to this actvity, many multi-story building have been erected within the area of the the new Management Centre of Naples (Fig. 12) in the last 15 years by using mixed solutions [23]. The high rise buildings from 50 to 100 m high have maily a structural system composed by reinforced concrete cores and steel skeleton (Fig. 8). The cores have the main structural function to resist the horizontal forces produced by earthquake or wind and they usually contain stairs and elevators. The surrounding skeleton, being simply pinned, is completely braced by the core and therefore its structural function is to resist the vertical forces only. The floor structures are usually made of both precasted concrete elements lightened with polystyrol and trapezoidal steel sheetings infilled with casted concrete. Beside to this current typology, also some special systems, always composite, have been conceived with innovative solutions; three of them merit to be mentioned [23].

First, the Law Court Building, composed by three towers, which are equal in plan but have different height varying from 78 to 177 m (Fig. 13). For each tower the structural system is composed by reinforced concrete curved walls, which provide strength and stability under horizontal loads, and a steel skeleton resisting vertical loads only. The floor slab is connected to the upper flange of beams by means of studs, giving rise to a composite horizontal diaphragm connecting the steel structure to the reinforced concrete walls.

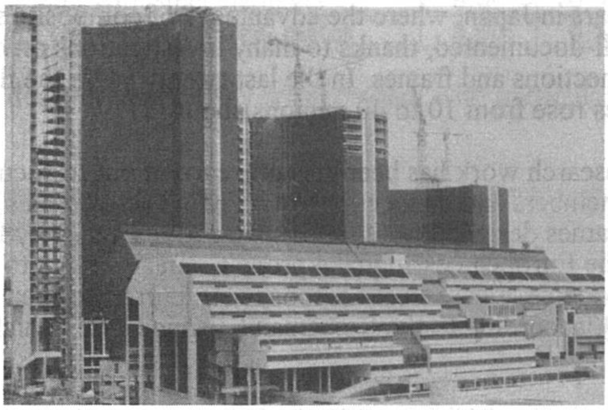


Fig. 13. The new Law Court building of Centre of Naples.

The second example is given by the two twin towers of the Electrical Department Headquarter (Fig. 14). Each tower has a lozenge shape 58x14 m and its structure is composed by two reinforced concrete cores connected at the top by a box-section girder which the 29 stories are suspended to. The suspended structure is made of steel ties and steel-concrete composite floor beams. The horizontal connections between cores and suspended structure are provided by means of elasto-plastic dissipative devices, which allow for a significant reduction of seismic effects, mainly at the base of the cores, where bending moment and shear are reduced of 30%.

Fig. 14. The National Electrical Department of Naples.

Finally, mention must be done to the main building of the new Fire Department, which is important at least for two reasons: first, because this building, initiated in 1981 and completed in 1985, was the first example in Italy of a base isolated structure; second, because it received the award of the European Convention for Constructional Steelworks in 1987. The structural scheme is based on a mixed structure [24], in which the concrete cores are spaced about 18x18 m and the steel skeleton is suspended to the top grid by means of vertical ties (Fig. 15). The top grid is connected to the upper part of the concrete cores by means of special devices, which isolate the steel skeleton from vertical and horizontal motions transmitted by the earthquake [25]. The bearing devices are made of a combination of rubber and teflon, which plays the double role to allow for free movements under serviceability conditions and to provide damping and energy absorbtion during an earthquake (Fig. 16).

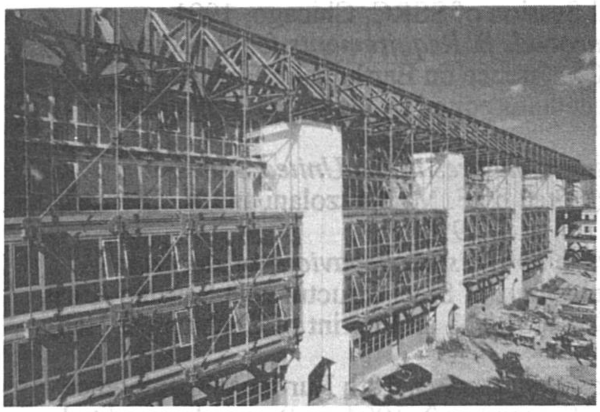


Fig. 15. The main building of the new Fire Department Centre in Naples.

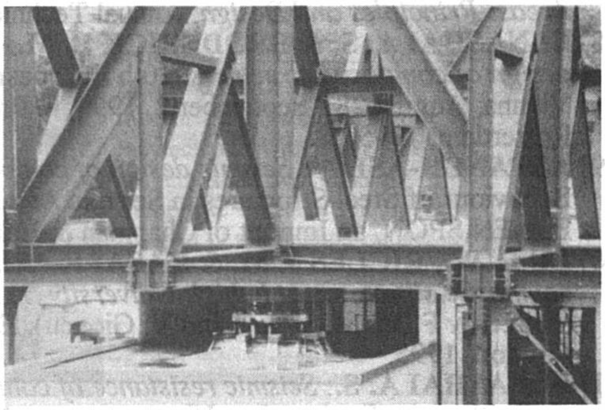


Fig. 16. Special devices to provide base isolation in the building of Fig. 15.

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Rehabilitation and Repair of Structures Using Composite Systems

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Summary

This paper presents a general and conceptual overview of the increasing importance of the rehabilitation and repair of existing structures that has been necessitated by growing demands and requirements of use. It calls attention to the positive features that both the direct application and derivations of composite construction can contribute to these types of activities. Finally, several unique cases of bridge modifications, such as the enlargement of platforms and the removal or translation of piers to increase low clearances, are introduced in order to help clarify the previous statements.

1. General overview

The incredible wealth that contemporary societies have in existing and utilized infrastructures and buildings, inherited from both historic and recent activity, is being subjected to degradation and aging with the passage of time, as well as to the loss of efficiency and safety against the growing demands of all types of activities: loadings, repetitive cycles, dynamic effects, etc., higher, in almost all cases, than the values considered during the design and construction of the work. This can be attested by the successive changes in codes and rules, even in the most recent periods.

The transformation of construction has been a systematic fact throughout history and there are many well-known cases in which a work was demolished in order to reuse its materials in the construction of a new, but conceptually different structure. At times this process was extreme, as can be seen in religious examples; but there are also cases in which the change was made for show or functionality or simply for urban renewal, prompted by the desires of a new invader, monarch or owner.

Today, this re-use of materials is non-existent. Our present construction materials, once dismantled, are of practically no value. Even to the point that a significant cost of transport to and storage at a rubbish dump must be taken into careful consideration in any demolition. And this comes before the negative ecological effect of this dumping is even considered, a problem which is gradually increasing and, will eventually make it necessary to find appropriate treatment methods for reducing its negative influence on the society.

For these and other reasons which will be introduced throughout this presentation, the best way to get the most out of this inheritance is to reuse the structural systems -not the materials- and to confer upon them new possibilities of resistance and improved durability and functionality that will adjust them to today's conditions and actively incorporate them in the process, thus avoiding the problems associated with demolition.

Rehabilitation conditions can be very wide and assorted, and therefore, although in general the total cost of the modification of a work tends to be cheaper, in many cases it can be equal to or even higher than the cost of the demolition and replacement of an older structure.

A number of factors such as the historic or artistic value of a building or bridge, its beauty or unique characteristics of its site; the reduction of nuisances that the preservation of the work would mean for the local residents and users (demolition; transport; noise; etc.); or even deeper, possibly ethical factors, which are of increasing importance due to the growing awareness of the intrinsic value of all the possible actions, can be determinants in the decision to choose an integral rehabilitation over the replacement of the work.

Furthermore, in the field of refurbishment or upgrading, higher costs, due to the greater construction periods and labor that this solution produces, can be a positive component for its selection in societies with high levels of unemployment, as it requires more manpower while reducing the basic energy costs of new materials.

In another sense, recently, new qualities and possibilities of a number of materials, techniques, equipment and processes of all types, which allow for a substantial improvement in the control, treatment and processing of methods used to facilitate and favor rehabilitation have appeared or been notably improved upon, independent of the similar advancements that new constructions techniques also provide.

This entire range:

- New and/or better materials to inject, replace or incorporate into existing structures.
- Improved knowledge of the true conditions of a work through precise control and detection equipment.
- Great precision and control of auxiliary systems, including monitoring that allows for the development of reliable processes in complex cases.
- Powerful equipment for all types of operations and processes;

significantly increases the potential of rehabilitation as a viable means of improving rather than replacing existing structures.

All of this, together with the previously mentioned, increasing social sensitivity to these methods, can lead to the conclusion that present conditions favor the development of this type of construction and that it is likely to become an important source of activity in many countries; especially when there is an historic component, that is, an important legacy to preserve in buildings and bridges.

From another side, apart from refurbishment or upgrading activities, other unexpected situations such as: earthquakes, impacts, and internal construction defects, can require the repairing or strengthening of structures or members. These activities must be done independent of economic criteria and, in general, in short terms and with guaranteed results.

The following presentation of the different topics or aspects that are a part of the modification of structures clarifies, in very broad terms, the importance of correct activity in this problematic field. For the most part, only an index with brief comments about the different possibilities which can be integrated in the modification of structures, will be shown. However, in the cases in which composite steel and concrete systems are used, some general ideas of the more common processes will be developed and then completed with a more detailed presentation of some unique solutions.

2. General lines of action

In general terms, the different fields that can reasonably be treated within the range of this presentation are:

2.1 Types of processes

The most generally used processes can be defined as the:

- Strengthening of structures and members to increase their: resistant capacity; fatigue resistance; stiffness against static or dynamic loadings; durability; etc.
- Repairing of structures damaged or affected by: impacts; overloading; intrinsic defects; etc.
- Rehabilitation, upgrading and adaptation of historic or even contemporary structures to better prepare them for today's demands such as: increased loading; repetitive forces; aggressive environment; etc.
- Enlargement of bridge platforms to increase their combined functionality and resistant capacity.
- Modification of a structure's supports to increase the clearance and functionality of its lower zones: translation or elimination of bridge piers or building columns; reduction of abutment areas; etc.

2.2 Materials

In lines of resistance, durability and construction, aspects of great importance in the selection of solutions and alternatives, a wide range of possible materials can be relied upon, which includes, but is not limited to:

- High strength concretes and mortars.
- Fluid mortars, without shrinkage or expansion.
- Mortars and other chemical products with high qualities of inalterability, strength, adherence, and fluidity for injection or substitution of cracked or damaged areas.
- High strength structural steels which are easily weldable but don't require special precautions because of their low carbon equivalent.
- Weathering steel with its high quality color and texture.
- Stainless structural steels with a wide variety of textures: glossy, matte, colored.
- Stainless steel re-bars for regions with very restricted covering thicknesses.
- Prestressing bars with a wide variety of qualities and diameters.
- External prestressing steels in single, self-protected strands.
- Advanced, high strength, composite materials: wires, cables, and sheets with unlimited durability and great lightness that balance their elevated price in risky cases, especially in areas which are highly susceptible to corrosion.
- Elastic bearing systems and shock transmission units (STU) for selected damping effects

2.3 Connection methods

Also with large number and variety of conditions which include, but are not limited to:

- Glues and adhesive resins for very strongly bonded connections between: concrete and steel; concrete and composite sheets; steel and composite materials; etc.
- Standard and specialized welds with low heat procedures.
- Very reliable chemical or mechanical bolts.
- Welded or mechanical stud and bolt shear connectors for standard carbon or stainless steels.
- High strength bolts (HST).

General procedures

Regarding the different types of structures and conditions to be considered for each solution, there are many different procedures which can be used, such as:

- Precambering by different methods: mechanical; thermal (pre-heating of the steel strengthening elements to be welded); prestressing (external and internal); etc.
- Preloading by support settlement.
- Load transfer between members by jacks.
- Lifting and transversal sliding for widening and pairing.
- Launching systems by pulling or pushing.

3. Steel and concrete composite systems

The basic goal of composite structures. is to achieve the maximum collaboration between steel and concrete and to exploit the best qualities of both in order to obtain a global system which is superior to the simple addition or juxtaposition of the materials.

The development of composite construction. the types and units for connection and the ways to reach this objective are extremely similar to the basic concepts used for the strengthening, repairing or rehabilitating of existing structures: full-employment of materials from the existing structure. or at least those which are still suitable for use. combined with the minimum quantities of additional materials. in order to achieve the fixed requirements of the final restored structure.

The behavior and all of the resources: formal. geometrical. resistant. links. joints. etc., of composite construction and project rehabilitation and repair clearly coincide and. solutions similar to new. contemporary composite structures. can be obtained directly for the repair and rehabilitation of existing works allowing for changes in the dimensions and arrangement of the materials. but with a nearly identical approach.

In fact. there are many. very well known. typical cases of the direct employment of composite systems for repair and rehabilitation. and to complete this presentation. first. several of these more systematic cases. will be briefly reviewed. Then. some more unique and special cases which open up the field and might stimulate in others ideas for the possible uses of new and favorable shapes in composite systems will follow.

3.1 Strengthening of concrete members by attaching thin steel sheets with resins. not only to their bottom faces but to the upper ones

It is helpful to complete these types of joints with mechanical or chemical bolts at the ends of the sheets to avoid local and progressive peeling due to: slight impacts. vibrations. curvature. end defects. etc.

Typical carbon steel sheets can be replaced by stainless steel ones to increase the durability: or by composites strips of organic materials: carbon fiber: etc. The latter are very easy to use because of their light weight and adaptability. but can cost more per kN of introduced tension.

3.2 Placement of flanges. bars or prestressing tendons in the bottom of the steel members of the structure. with the possibility of eventually combining these with the placement of new or the improvement of existing upper concrete slabs. in order to increase the global capacity.

In general. the joining and anchoring of these elements is relatively simple and the only real difficulty lies in the capacity of the existing shear connections between the tension and compression reinforcement. which can be problematic. or even a reason to discard certain types of solutions.

3.3 Total or partial wrapping of concrete columns or piers in order to create a composite member with greater axial resistance and. more importantly. with a higher shear capacity to improve the performance of these members in seismic areas or in areas were they are susceptible to impacts.

Shear connections by studs or. preferentially. by bonded resins combined with hoop stresses due to the welding or pre-heating shrinkage of the partial elements which wrap the concrete shaft, can be use to obtain the final element or member.

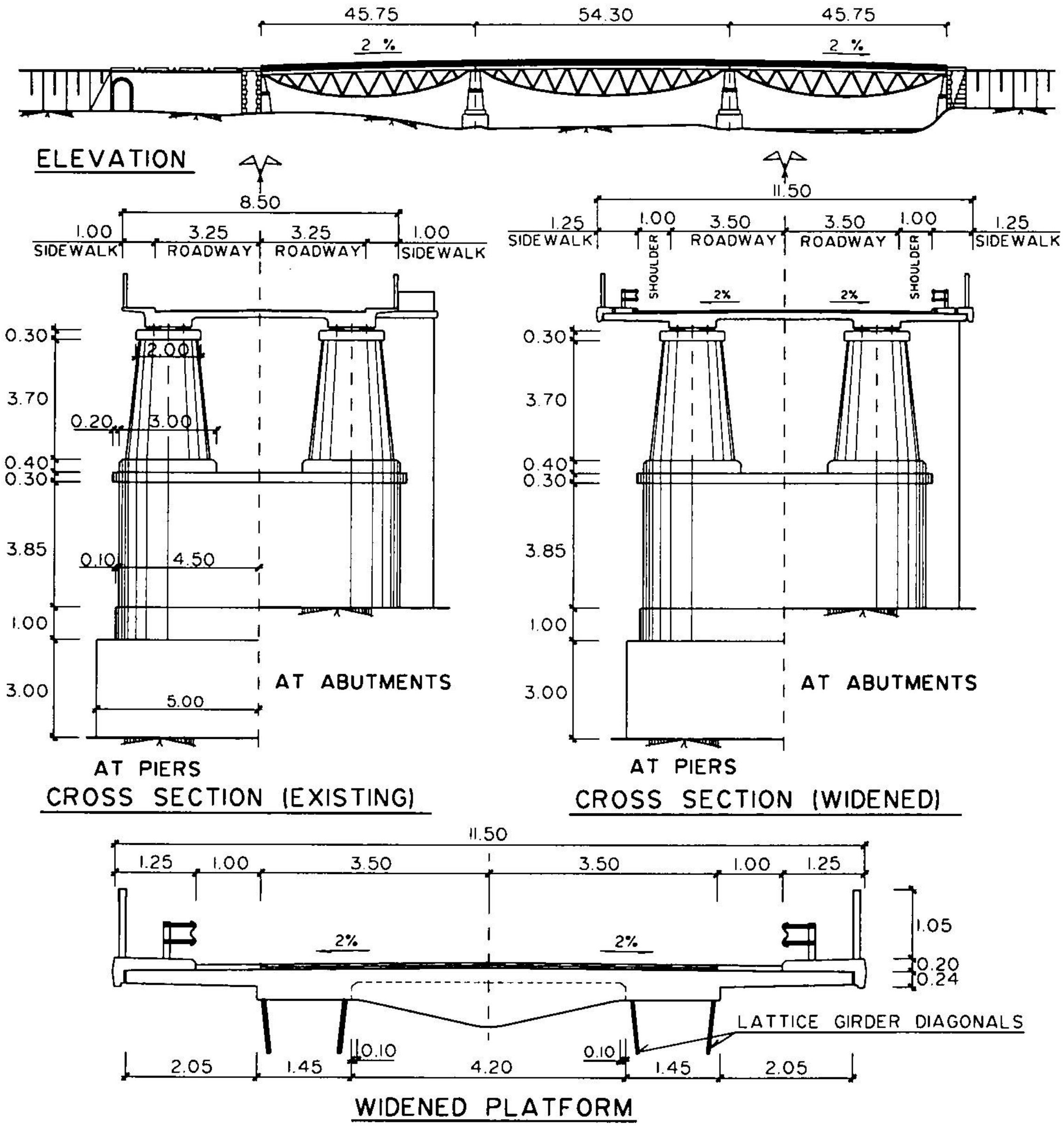
4. Unique systems

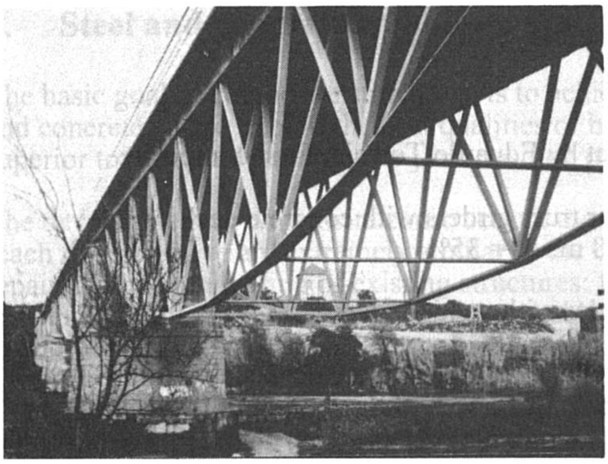
4.1 Widening of bridge platforms

Bridge over Tordera River (Barcelona). 1940. Project by Eduardo Torroja. First composite bridge in Spain.
3 isostatic spans of 46-54-46 m. Twin, steel funicular truss girders with concrete deck.
Original platform width: 8.50 m. Final weight: 11.50 m. $\Delta_b = 35\%$
<u>Procedure</u>

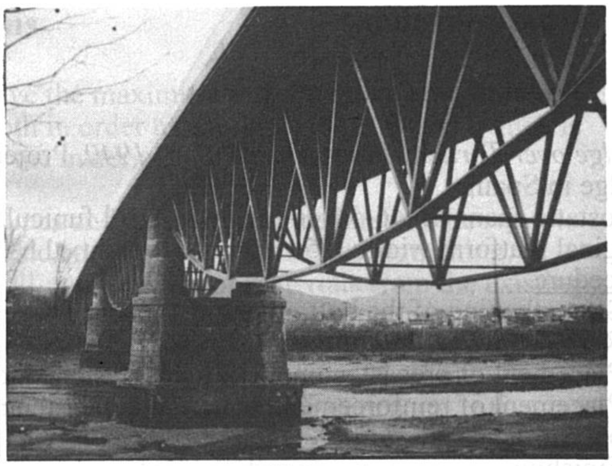
- Strengthening of the bottom chord of the truss girder with welded plates.
- Removal by cutting of the existing cantilevers.
- Placement of precast, composite slabs, fastened to the existing concrete deck slab with HSTs.
- Placement of reinforcement, concreting over the precast slabs, and thickening of the central part of the deck.
- Finishing.

<u>Final structural behavior</u>
Complete, composite collaboration of the older system and the concrete newly incorporated into it.

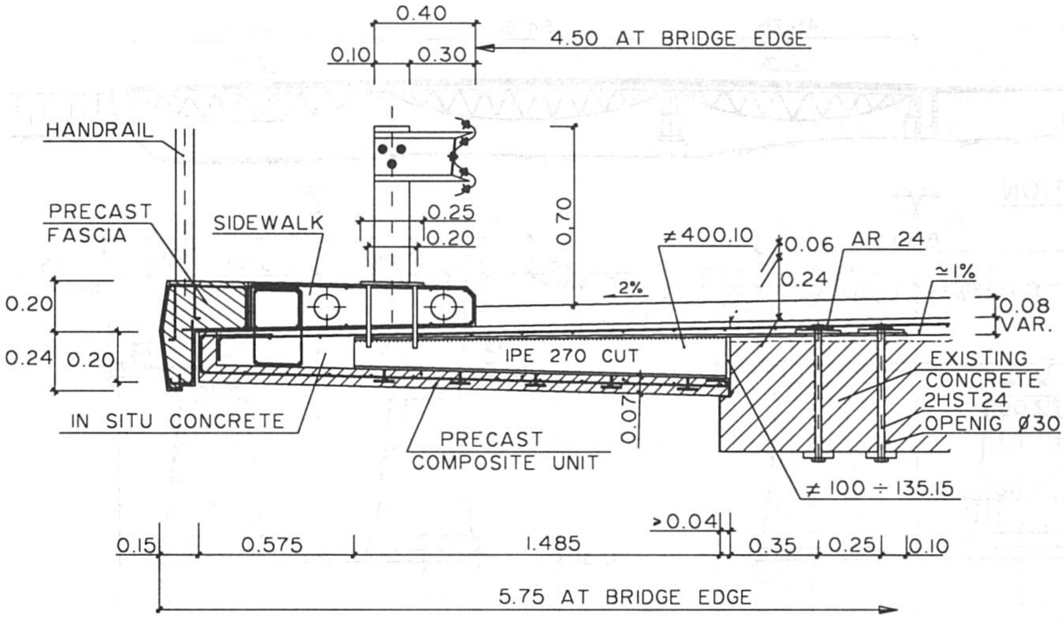




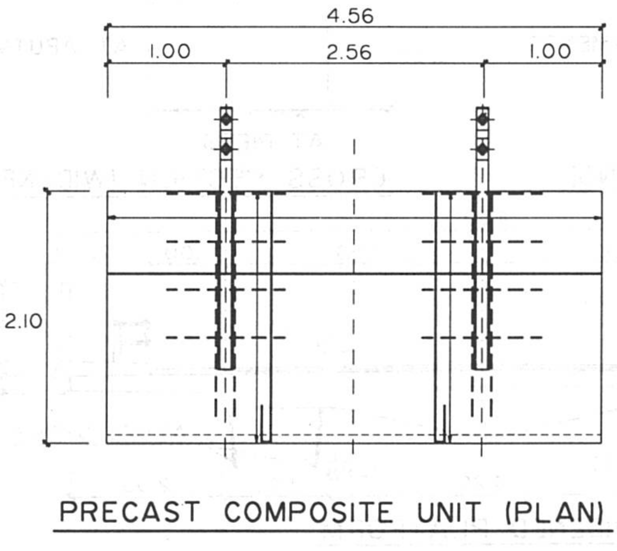
Existing structure



Widened structure



CANTILEVER WIDENING DETAIL



PRECAST COMPOSITE UNIT (PLAN)

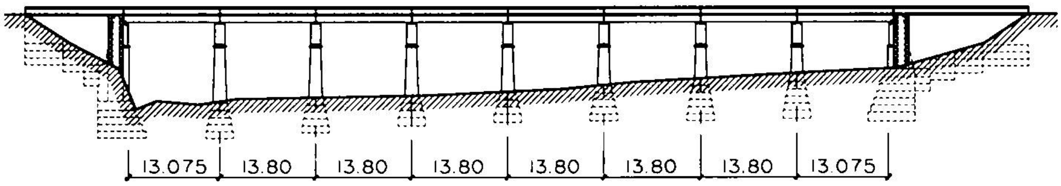
Bridge over Asma River (Tarragona). 1910. Eight single spans of 13.80 m. Reinforced π shape concrete deck platform.
Original platform width: 6.20 m. Final width: 10.50 m. Δ_b = 69.4%

<u>Procedure</u>

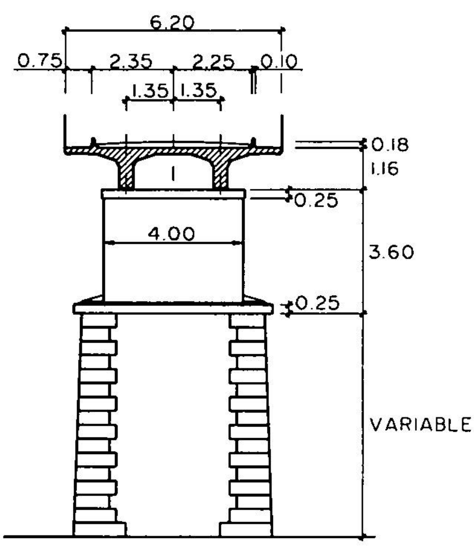
- Removal by cutting of existing cantilevers.
- Placement of weathering steel box girders that are attached with fasteners to the concrete Π member.
- Placement of precast composite slabs anchored to the steel girders.
- Placement of reinforcement, concreting over the precast slabs and thickening of the central part of the deck.
- Finishing.

<u>Final structural behavior</u>

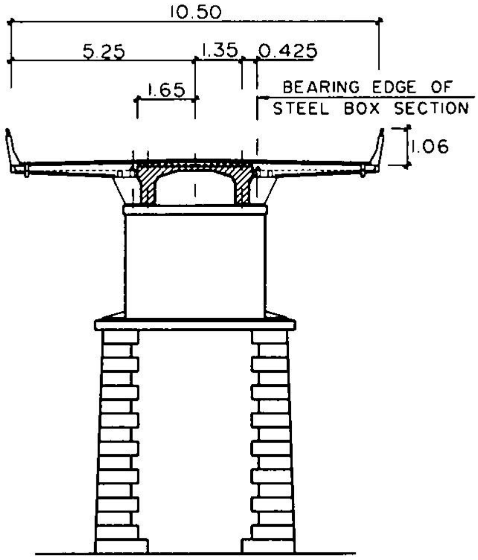
Completely composite structure with collaboration between the new concrete slab; the older π shaped slab, the reinforced concrete beam; and the steel box girders.



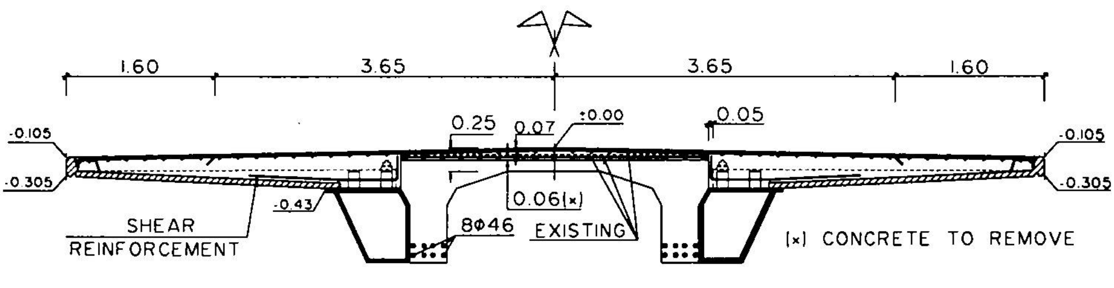
ELEVATION



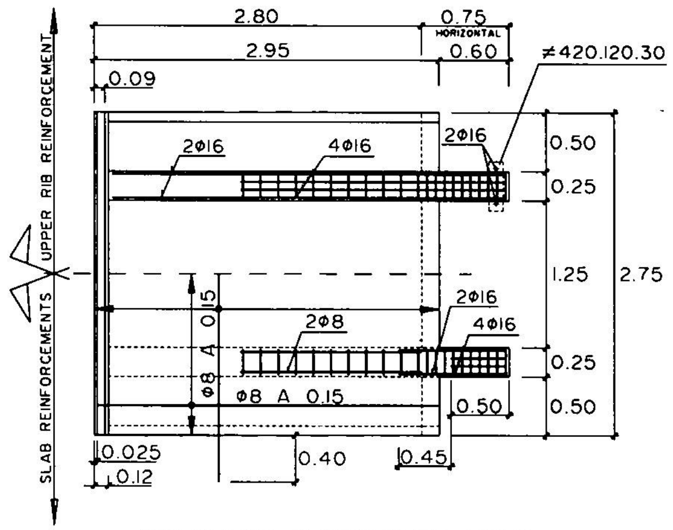
CROSS SECTION (EXISTING)



CROSS SECTION (WIDENED)



TRANSVERSE CROSS SECTION (REINFORCEMENT IN SITU CONCRETE)



PRECAST CONCRETE UNIT

4.2 Amplification of spans of overpasses

14 Overpasses in the A7 Highway Barcelona-France. 1965. Prestressed concrete deck slab with cantilevers, lightened by circular openings.
Original spans: 12.50 - 15.25 - 15-25 - 12.50 m.
Final spans: 8.95 - 18.80 - 18.80 - 8.95 m. ΔL = 23.3%

The solution is based on the incorporation of two weathering steel box girders placed parallel to the deck slab under its overhang. Like a stretcher, the system transfers the loads from the existing piers, to new ones.

Originally, several other solutions were analyzed, some with external prestressing, others which included modifications of the abutments, etc., but the cost, period, erection and, especially, the resulting aesthetics, determined the final solution chosen.

Procedure

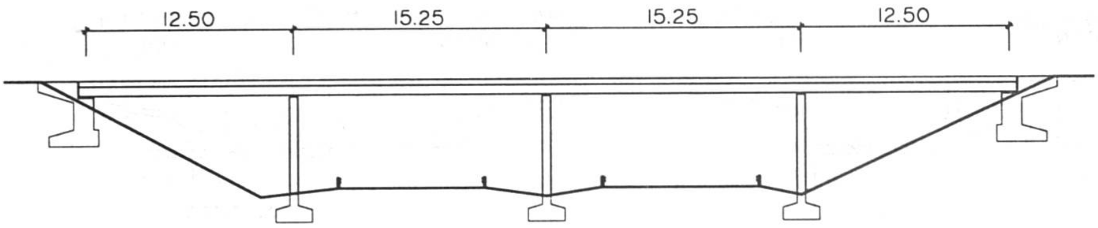
- Amplification of unaffected piers by connecting hollow steel shafts to them.
- Erection of the new, composite piers, which appear identical to the previously widened, unaffected ones.
- Transverse drilling of the concrete deck slab over the piers to be eliminated, in order to introduce the tying prestressing bars.
- Placement of elastic, neoprene bearings over the new piers and in contact with the bottom of the concrete deck.
- Placement of the steel box girders, first with cranes and then by sliding them over teflon strips.
- Transfer of the loads from the piers to be removed onto the box using groups of hydraulic jacks. The initial position of the concrete deck slab remains unchanged. The precambered steel girders lose their curvature and their bottoms remain parallel and at the level of the deck slab soffit.
- Connection of the steel girders to the deck slab with prestressing bars.
- Removal of jacks.
- Load testing of the bridge.
- Demolition of old piers.

Essential aspects

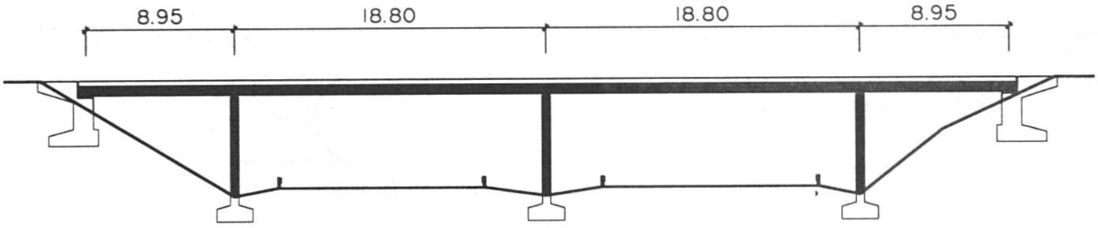
Neither the traffic on the highway nor on the overpass was disturbed.

Final structural behavior

The concrete deck slab and the steel box girder work together to transfer the loads to the bearings. The elastic bearings at the new piers restrict the hogging bending moments of the concrete slab at these points to the maximum internal capacity of the affected cross sections.

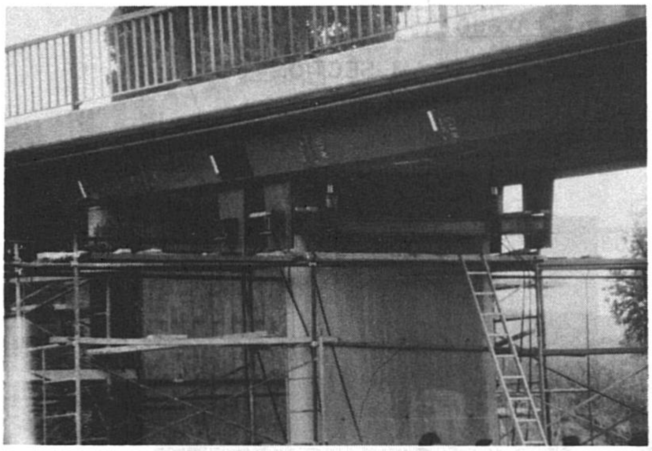


ORIGINAL ELEVATION

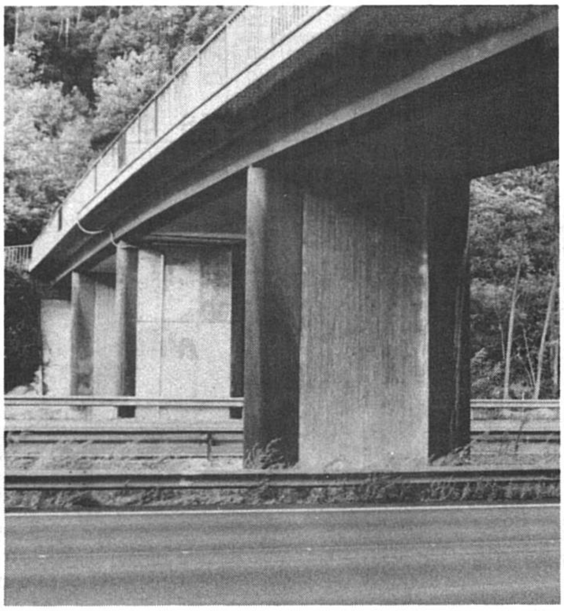


FINAL CLEARANCES

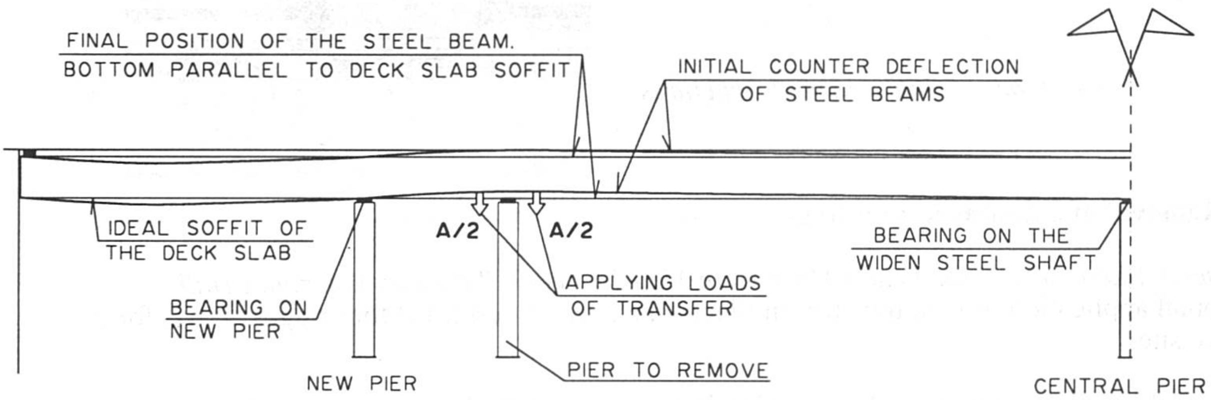
LAYOUT OF TYPICAL OVERPASSES



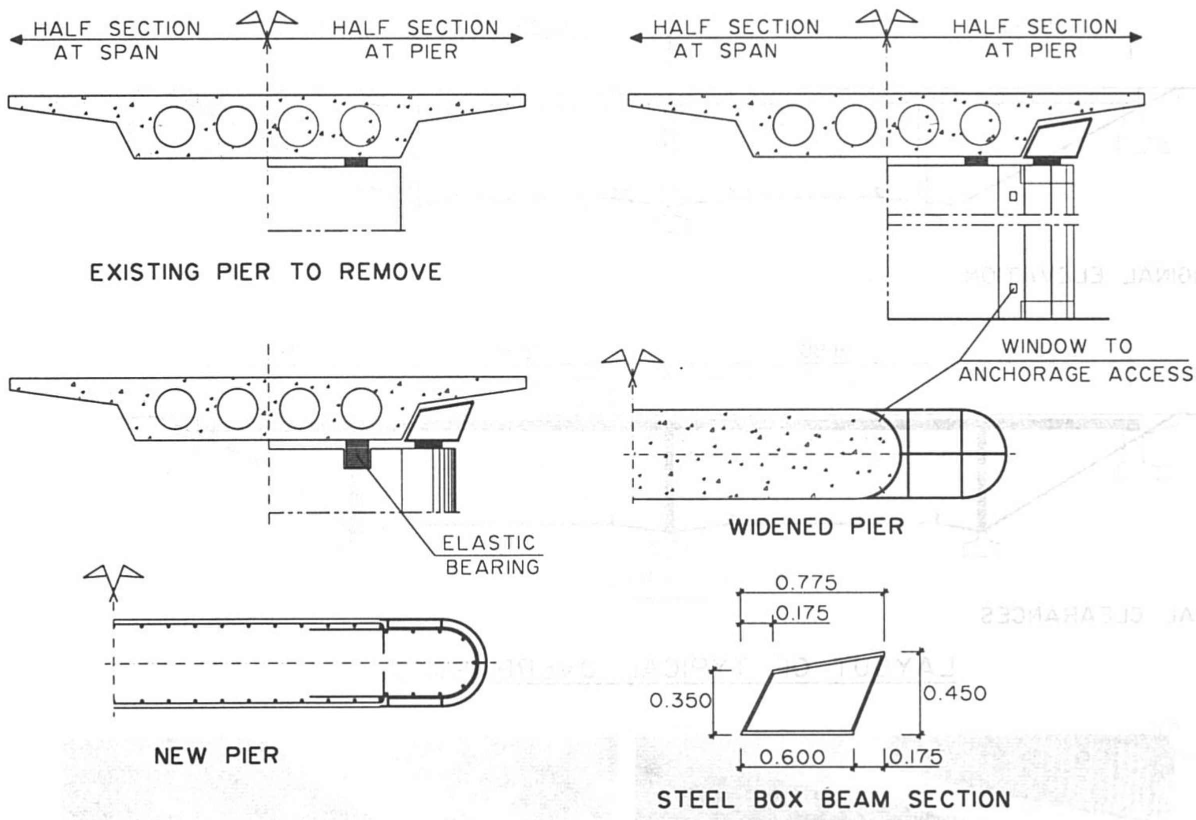
Transfer of loads



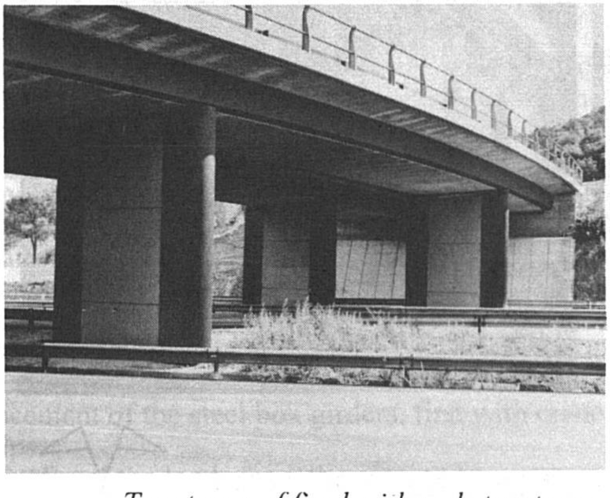
Final structure

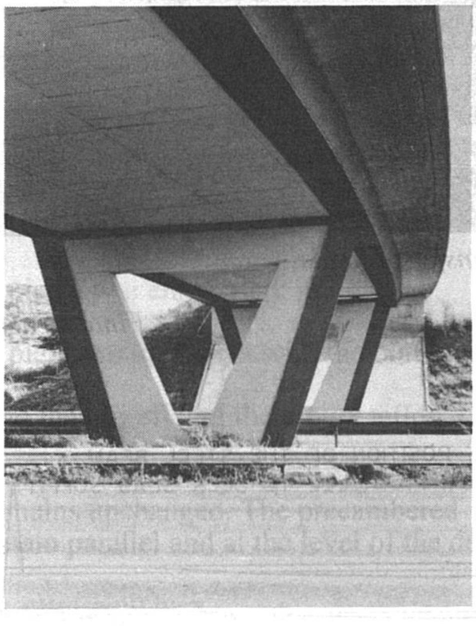


COUNTER DEFLECTIONS AND FINAL PLACEMENT OF THE STEEL
BOX BEAMS AFTER THE TRANSFER OF LOADS



FEATURES OF CROSS SECTIONS





Two types of final widened structures

4.3 Removal of columns in buildings

Villanueva Auditorium at the Prado Museum in Madrid, below Paintings Velázquez Hall
Conceptual application of load transfer, similar to the process described for bridges above, for a sensitive site.

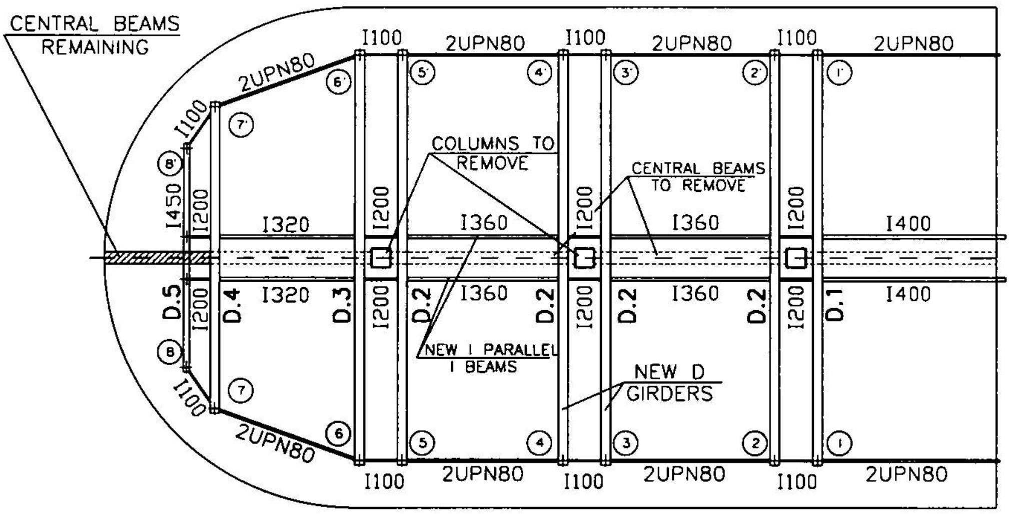
A system of parallel beams was placed under the existing longitudinal beam and adjacent to the columns to be removed, so that once they were eliminated the newly placed beams filled the gap that was left by the original.

Procedure

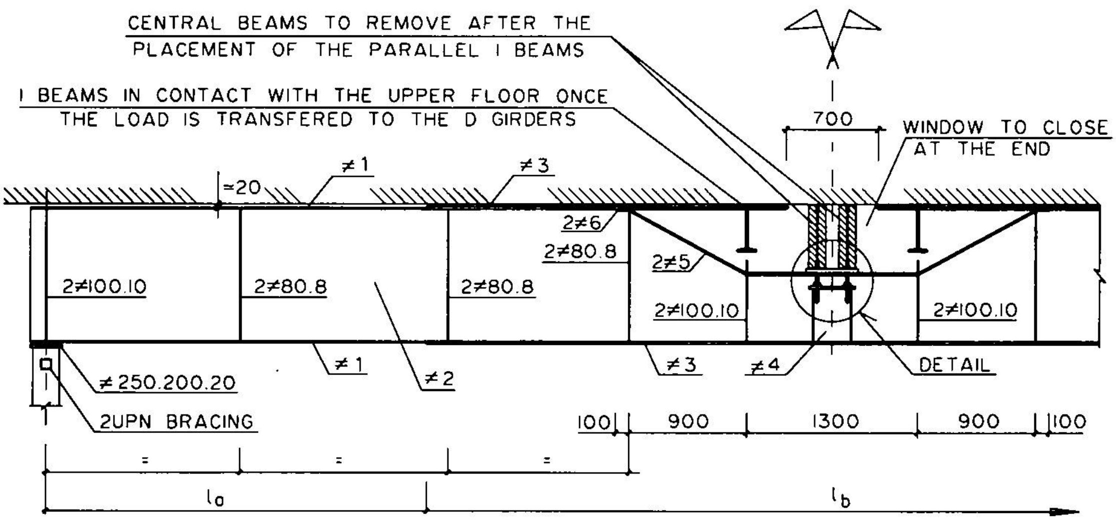
- Placement of pairs of new transverse girders.
- Placement of plates and bolts for transfer.
- Turning of bolts with gradual transfer of the column's loads to the new girders, which are free to deflect, without changing the position of the original structural system.
- Placement and welding of a pair of ancillary beams, parallel to the existing central one.
- Removal of the original central beam.
- Removal of the columns.
- Closing of the central holes in the transverse grids.
- Finishing.

Final structural behavior

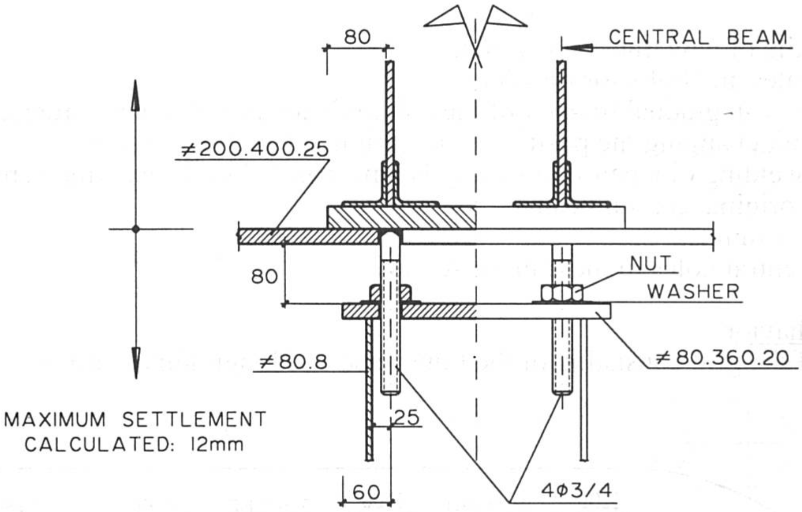
Combined work of the grid consisting of the transverse and longitudinal girders.



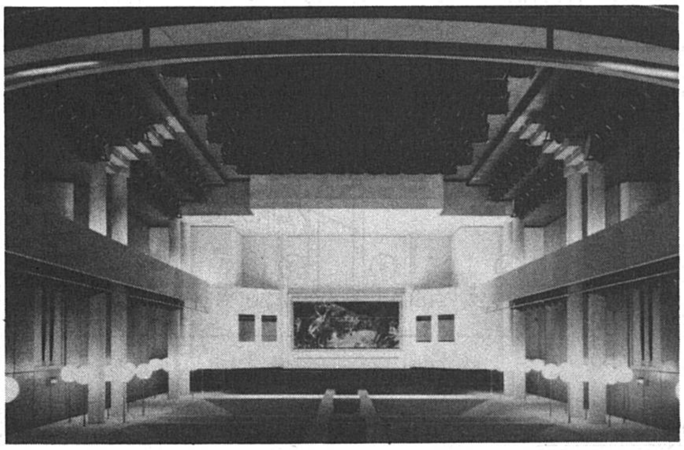
NEW SUPPORTING STRUCTURE OF VILLANUEVA AUDITORIUM



NEW GIRDER D



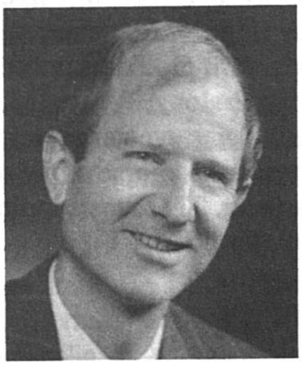
DETAIL: TRANSFER DEVICE BETWEEN CENTRAL BEAMS AND NEW TRANSVERSAL D GIRDERS



Final Auditorium

Repair Using Advanced Composites

Urs MEIER
Professor
EMPA
Dübendorf, Switzerland



Urs Meier, born in 1943, received his Engineering degree at the Swiss Federal Institute of Technology, ETH, in Zurich. In 1971/72 and 1977/78 he was Research Associate at the Massachusetts Institute of Technology, MIT in Cambridge, USA. Since 1971 he has been involved in research and development on advanced composites and he is now Director of the Swiss Federal Laboratories for Materials Testing in Dubendorf. He is also Professor at the ETH Zurich.

Summary

This paper discusses the advantages of using advanced polymer matrix composite materials, originally developed for high-performance aircraft, for post-strengthening existing structures. Criteria for evaluating and designing with these materials are suggested. In Switzerland, retrofitting by externally bonding carbon fibre reinforced plastic (CFRP) laminates has been shown to be less expensive than the technique of external steel plate bonding, especially if ease of handling is a dominant cost factor.

1. Introduction

Changing social needs, upgrading of design standards, increased safety requirements and deterioration result in existing structures that need to be retrofitted or demolished. Many existing structures are part of the architectural heritage and demolition is not a viable option. Even for newer structures, rehabilitation is in most cases a much better use of resources than replacement. Bridges represent a major proportion of engineering structures. They are a significant factor in the infrastructure and their maintenance has implications on the economic life of a nation through disruption and traffic delays.

Chloride induced deterioration of reinforced and prestressed concrete bridges, continual upgrading of service loads and the large increase in the volume of traffic means that thousands and thousands of bridges need repair or reconstruction. This paper will demonstrate the use of advanced composite materials such as thin carbon fiber laminates bonded to existing structures to strengthen and rehabilitate them to extend their useful life. Approximately 5% of the deteriorated bridges in Europe can be strengthened using advanced composite materials instead of conventional steel plate bonding. A further 5% can be saved from demolition by this method. The saving in Europe will be in the region of 5.5 Billion US $ per annum. In addition there are savings on other structures in the need of strengthening. These savings are less easy to quantify in Europe. However, the potential on "other structures" is at least similar. If we estimate the annual worldwide potential it will be at least 10 Billion US $. 20 Billion US $ may be much closer to the reality.

2. Why Should We Replace Steel Plates?

Today in Western Europe and in other parts of the world the strengthening technique for bonding steel plates is wide spread and is the state of the art. In a non-corrosive environment this technique shows very good long-term behaviour (Fig. 1). However, weathering tests over extended periods of time have indicated that long term problems concerning corrosion of the steel must be expected in outdoor applications. Ladner, Pralong, Weder [1] observed "small traces of rust" on unprimed as well as primed bonded steel plates even after only 3 years exposure to weathering. The rust became more extensive during the course of the test and after 15 years exposure the areas have grown to 10 mm in diameter. These tests indicate a weakness in the strengthening of structural systems using steel plates [2].

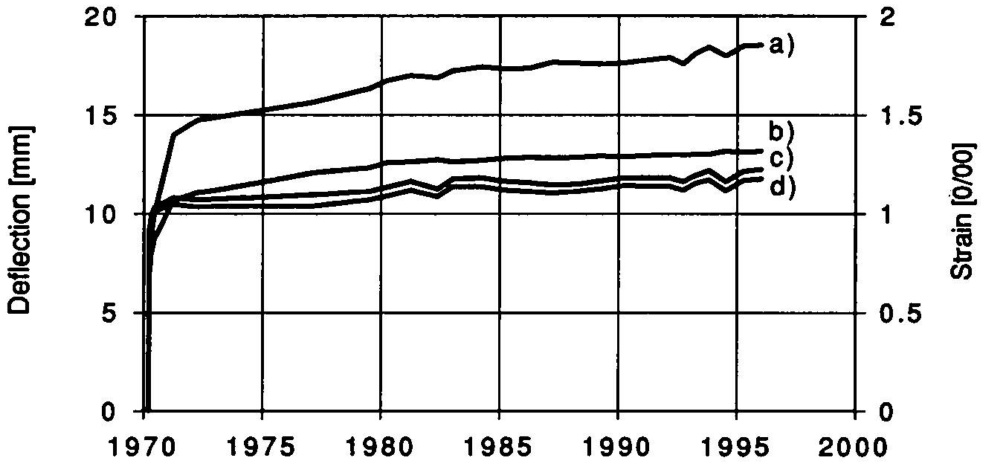


Fig. 1 Creep data for 26 years of a RC girder (190-depth x 210 x 2895 mm) post-strenghthened with a steel plate (7 x 120 x 2820). The steel plate is bonded to the bottom of the beam using a filled epoxy resin (Ciba), without mechanical fasteners. The beam is loaded in 4-point bending (945 mm shear spans) using 30 kN lead weights at each of the two loading points. The induced, constant bending moment is 28.35 kN·m. The temperature variation during the indoor test is 16 to 24°C, and the range of relative humidity is 40 to 80%. The curves represent

a) compressive strain of the concrete at the top of the girder [0/00]
b) deflection at mid-span [mm]
c) tensile strain of the steel plate [0/00]
d) tensile strain of the concrete at the steel plate interface above (c) [0/00].

Steel plates have other disadvantages. During renovation work, particularly on bridges, generally only a limited amount of mechanized lifting machines are available. In the interior of box girders, for example, the heavy strengthening plate has to be carried by hand to the point of installation.

Consequently, due to handling limitations on site, the steel plates are rarely longer than 6-8 m; however, if the strengthening work involves greater plate lengths, a butt-joint system must be used. This type of joint cannot be welded since the welding temperatures would destroy the adhesive bond, consequently butt-jointed steel plates have to be formed from single shear lap joints. If steel plates were replaced by high strength fiber composites a relatively thin component could be delivered to the building site in rolls of lengths in excess of 300 m. Compared to steel plates their bonding technique is greatly simplified. Using bonded CFRP laminates [3] the quality assurance can be demonstrated by infrared inspection in the field, as discussed in a later section; this is not possible with steel plates.

3. Which Is The Most Suitable Fiber?

The partial substitution of steel plates with polymer matrix/fiber composites was discussed in Europe in the early eighties. One of the most important decisions which had to be addressed at that time was that regarding the most suitable fiber composite material for this application; Table 1 lists criteria which specifically relate to the use of composite materials as a post strengthening material for structures and applies particularly to prestressed laminates. These criteria may not necessarily satisfy other applications. The ratings in Table 1 are rather crude, however, it is clear that the tensile strength is a relevant criterion, but the significance of the compressive strength may be questioned as concrete generally has to be strengthened in the tensile region of the beam. In certain static systems, however, there may be regions which are normally stressed in tension but which may also be subjected to compressive stress depending upon the load distribution; in these situations bonded steel plates are not acceptable as they will peel off. Aramid fiber reinforced polymers would also fail due to their poor compressive strength. Deuring [4] has shown that carbon fiber reinforced composites (CFRP) do satisfy the compressive requirement.

Table 1: Quantitative Rating Of The Fiber Types

Criterion:	Weighting Factor:	Weighted Rating For Laminates With Fibers Of:		
Range Of Weighting Factor	1 ... 3	Carbon	Aramid	E-glass
Tensile Strength	3	9	9	9
Compressive Strength	2	6	0	4
Young's Modulus	3	9	6	3
Long-Term Behavior	3	9	6	3
Fatigue Behavior	2	6	4	2
Bulk Density	2	4	6	2
Alkaline Resistance	2	6	4	0
Price	3	6	6	9
Total Points		55	41	32.
Ranking		1.	2.	3.

Rating: very good = 3, good = 2, adequate = 1 and inadequate = 0 points

The modulus of elasticity of the laminate material is of great significance when the laminate is not prestressed before being bonded, because only stiff laminates are able to relieve the stresses in the existing internal steel reinforcement. Laminates fabricated from glass fiber reinforced composites (GFRP) must be 4-10 times thicker than CFRP laminates to achieve the same tensile stiffness. If such GFRP composites are longer than 6-10 m their handling on the construction site is difficult.

The fatigue behavior of the system may be important or insignificant depending on the structure and the nature of the loading. The bulk density of the material is less important as a criterion since the density of all the fiber composites considered is low compared to that of steel.

The cost criteria is important. If a comparison of the price of fiber composites is made with that of the standard steel Fe 360 then it would appear, at first glance, that fiber composites are far too expensive. The price factor based on unit volume of material is 4 to 20 times greater than that of steel. However, when the cost of upgrading the structure is considered, the material cost amounts to less than 20% of the total cost of the construction, consequently, when the ease of handling of the fiber composite system is considered the solution becomes competitive due to its light weight.

4. Conclusions Regarding The Material Evaluation

From the above mentioned considerations result the following conclusions:

(i) The applications in which corrosion plays no role and the length of the strengthening component is less than 5 m, steel will be the favorite material; this is the case mainly for building construction. As will be shown later, however, laminate thickness may play a role from the point of view of aesthetics; thus interior decoration and non-technical considerations lead to renovation solutions with thin fiber composite laminates rather than plates.

(ii) In applications where corrosion, length of strengthening component and handling on construction sites play dominant roles, for example bridges, multistory parking spaces, railway stations and specialized industrial structures, historic monuments, fiber composites must be considered seriously.

(iii) In applications such as slabs with fire sprinkler systems, the pipe installation would have to be removed in order to bond heavy steel plates on to the under-strength units. Composites, however, would be able to compete with steel successfully as the thin strengthening components would be bonded insitu. Consequently, labor costs would be reduced substantially and the fire protection system of pipes and outlets would be operational during rehabilitation work.

(iv) The results shown in Table 1 clearly indicate that carbon fiber reinforced polymer composites most closely fulfill requirements for the post strengthening of structures. Consequently, from the early eighties laboratories in Europe have concentrated their research efforts using this material and all further discussion will be restricted to carbon fiber reinforced polymers. Typical properties of the composites considered are given in Table 2. The large scale research project undertaken in 1993 in the USA and in Canada in the area of carbon fiber/polymer composites and many successful applications in Japan have confirmed the earlier Swiss decision to use this material in construction.

5. Strengthening with Unprestressed CFRP Laminates

The research work shows the validity of the strain compatibility method in the analysis of various cross sections [4-6]. This implies that the calculation of flexure in reinforced concrete elements which are post strengthened with carbon fiber reinforced epoxy resin composites can be performed in a similar way to that for conventional reinforced concrete elements. The work also shows that the possible occurrence of shear cracks, may lead to peeling of the strengthening composite. Thus, the shear crack development represents a design criterion. Flexural cracks are spanned by the CFRP laminate and do not influence the loading capacity. In comparison to the unstrengthened beams, the strengthening laminates lead to a much finer cracking distribution. A calculation model [5] developed from the CFRP composite agrees well with the experimental results.

Table 2: Properties Of Laminates

Fiber Type: → Property: ↓	T 300	T 700	M 46 J
Fibre Volume Fraction [%]	70	70	70
Longitudinal Strength [MPa]	2000	2800	2600
Longitudinal Elastic Modulus [GPa]	148	152	305
Strain At Failure [%]	1.4	1.8	0.85
Density [g/ccm]	1.5	1.5	1.6

When a change of temperature takes place the differences in the coefficient of thermal expansion of concrete and the carbon fiber reinforced epoxy resin composites result in thermal stresses at the joints between the two components. After 100 frost cycles ranging from + 20 degree C to - 25 degree C, no negative influence on the loading capacity of the three post-strengthened beams was found [5].

For the post strengthened beams the following failures were observed:

(i) The CFRP composite failed during loading with a sharp explosive snap, the impending failure was preceded well in advance of the failure by cracking sounds, concrete cracking and large deflections.

(ii) Classical concrete failure in the compressive zone of the beam.

(iii) Continuous peeling-off of the CFRP laminates due to an uneven concrete surface. For thin laminates of thickness less than 1 mm and bonded to the concrete surface with the aid of a vacuum bag, an even bonding surface is required. If the surface is too uneven, the laminate will slowly peel off during the loading.

(iv) Shearing of the concrete in the tensile zone (it can also be observed as a secondary failure).

(v) Interlaminar shear within the CFRP laminate (observed as secondary failure).

(vi) Failure of the reinforcing steel in the tensile zone (this failure mode was only observed during fatigue tests).

The following failure modes were not observed but are theoretically possible;

(i) Cohesive failure within the adhesive.

(ii) Adhesive failure at the interface between the CFRP laminate and the adhesive.

(iii) Adhesive failure at the interface between the concrete and the adhesive.

For post strengthening with CFRP composites it is recommended that the design rule for the CFRP composite is such that it should fail during yielding of the steel reinforcing bars before a compressive failure of the concrete. Yielding of the steel bars should not occur before reaching the permitted service loads.

Kaiser [5] investigated a 2 m span beam under fatigue loading. The cross-section was 300 mm wide and 250 mm deep. The existing steel reinforcement consisted of 2 rebars of 8 mm diameter in the tension and in the compression zones. This beam was post strengthened with a glass/carbon fiber hybrid composite having the dimension 0.3 by 200 mm. The fatigue loading was sinusoidal at a frequency of 4 Hz; the test set up corresponded to a four point flexure test with loading at the one third points. The calculated stresses in the hybrid laminate and the steel reinforcement are listed in Table 3. After 480'000 cycles the first fatigue failure occurred in one of the two reinforcing rods in the tension zone; after 560'000 cycles the second reinforcing rod failed at another cross-section; after 61'000 cycles a further break was observed in the first reinforced rod and after 720'000 cycles a second break in the second rod was observed. The first damage to the composite appeared after 750'000 cycles and it was in the form of fractures of individual rovings of the laminate; the beam exhibited gaping cracks, which were bridged by the hybrid laminate. The relatively sharp concrete edges rubbed against the hybrid laminates at every cycle and after 805'000 cycles the composite finally failed, however, the test was executed with unrealistically high steel stresses. The aim of the test was to gain insight into the failure mechanism after a complete failure of the steel reinforcement; it was surprising to observe how much the hybrid laminate could withstand after failure of the reinforcement.

Table 3 Exaggerated Fatigue Loading And Corresponding Stresses

Loads [kN]	Stresses [MPa] Rebars	Laminate
Minimum 1	21	11
Maximum 19	407	205

Table 4 Realistic Fatigue Loading And Corresponding Stresses

Loads [kN]	Stresses [MPa] Rebars	Laminate
Minimum 125.8	131	102
Maximum 283.4	262	210

Deuring [4] performed further fatigue tests on a beam with a span of 6 m under realistic loading conditions. The total load carrying capacity of this beam amounted to 610 kN without post strengthening. When the beam was strengthened by bonding a CFRP composite laminate, with dimensions of 200 x 1 mm (laminate type T 300, Table 2), its load carrying capacity was increased by 32% to 815 kN. The calculated stresses in the CFRP laminate and the steel reinforcement are given in Table 4. The beam was subjected to this loading for 10.7 million cycles. After 10.7 million cycles the tests were continued in an environmental condition where the temperature was raised from room temperature to 40-degree C and the relative humidity to a value of 95% r.H. The aim of this test was to verify that the bonded CFRP composite could withstand very high humidity under fatigue loading. Initially the CFRP composite was soaked with water to nearly 100% saturation. After a total of 12 million cycles the first steel reinforcement failed due to fretting fatigue. The joint between the CFRP laminate and the concrete did not present any severe strain fatigue. In the next phase of the test program, the external loads were held constant (Table 4) and the stresses in the reinforcing steel and the CFRP laminate decreased. After 14.09 million cycles the second reinforcing steel rod failed, also due to fretting fatigue. The cracks which were bridged by the CFRP composite laminate rapidly grew and after failure of the third reinforcing rod, due to yielding of the remaining steel, the CFRP laminate was sheared from the concrete.

6. The Effect of Lightning or Fire on CFRP Laminates

The destructive effects of lightning are well known. The studies of lightning and the means of preventing its striking an object or the means of passing the strike harmlessly to ground have continued since the days when Franklin first established that lightning is electrical in nature. From these studies, two conclusions emerge; firstly, lightning will not strike an object if it is placed in a grounded metal cage and secondly, lightning tends, in general, to strike the highest objects in the area. As composite materials replace more and more metals in aircraft, there has been an increase of risk of damage by lightning to such composite sections. CFRP is a conductor, but is relatively resistive to electricity which causes it to heat up as the current passes through it. A lightning strike has two main effects on unprotected CFRP; firstly, the main body of the CFRP becomes so hot that the epoxy resin component vaporizes and secondly, the structural integrity of the CFRP will have been affected after the carbon cooled down. It will probably retain a considerable tensile strength but it will loose interlaminar shear and compressive strength. Therefore, the aircraft industry developed aluminum grids which are used to protect the composite in its outermost layers.

In most applications in which CFRP laminates are used for strengthening, they are not exposed to lightning strikes as they are inside a building or box girder which is equivalent to grounded cages. Composite laminates used in bridge strengthening are positioned on the soffits of the beam and lightning will have no access to them in this case. If there are situations where lightning may be a danger, metal grids, which are used with composites in aircraft design, have to be utilized.

In 1994 the EMPA performed a series of bending tests on strengthened beams positioned in a large horizontal testing oven [8]. The span of the 6 beams tested was 5.2 m and their width and depth were 400 mm and 300 mm respectively; the volume fraction of the steel rebars was 0.65%. The beams were loaded by hydraulic activators with the maximum short time load as laid down in the Swiss code SIA 160 (1989) in four point bending. The oven was heated according to the ISO Standard 834 with a temperature of 925 K after 1 hour. One beam was not plated and acted as the control, another beam was post strengthened with steel plates (75 mm wide, 8 mm thick) and in addition four beams were post strengthened with CFRP laminates (74 mm wide, 1 mm thick). After 8 minutes duration of the test the steel plate came away from the beam. During the test in which the beam were post strengthened with CFRP laminates, the fibers started to burn at the surface of the laminates and their cross sections slowly decreased in value, thus causing a slow decrease in stiffness. The CFRP composites finally became unbonded from the beam after one hour. The main reason for the superior behavior of the CFRP composites compared with that of the steel plates was their low thermal conductivity in the lateral direction.

7. Safety Considerations in the Case of the Lightning and the Fire

Since the early seventies it was [1] always recommended that the post strengthening of a structure should not be more than 50%. Therefore, after an accidental failure of the post strengthening of the beam, a residual factor of safety of approximately 1.2 would remain and the collapse of the structure could be avoided.

8. Quality Assurance

In Switzerland the pulse thermography (infrared inspection) [8] is applied for quality assurance of the bonding of the CFRP laminates to the structural surface. This non-destructive testing method relies on changes in thermal conductivity caused by flaws or damage. The equipment used for infrared inspection is currently small and light weight, thus allowing analysts to gather more sophisticated information regarding the object being tested on line on the construction site. The technique operates on the principle that an infrared camera is positioned in front of the laminate which is heated with a flash lamp. The sensors in the infrared camera detect the heat that is absorbed and then re-radiated from the surface and the digitized information is sent to a video board in a computer from where an image is constructed on a video screen. This image allows fast and accurate judgment on the quality of the strengthening work.

9. Applications in Europe

To the best knowledge of the author the Kattenbusch Bridge in Germany is the first place in the world where fiber reinforced plastic laminates were used to strengthen a bridge. After World War II numerous prestressed concrete bridges for motor vehicles were built in Germany employing the method of in-situ spanwise construction. These continuous multispan bridges are mostly designed as box girders. The working joints are at the points of contraflexure where usually all of the tendons are coupled. Many of the bridges now exhibit cracks at the working joints. Usually, the bottom slab of the box girder is transversely cracked at the joint. This relatively wide crack grows into the webs with diminishing width. Thereby it crosses the lower tendons and couplings. The main cause of these cracks is a temperature restraint which was not taken into account during previous designs [9-11]. In combination with other stresses tensile stresses at the bottom increase and exceed the concrete tensile strength at the joint. As the reinforcement ratio of the bottom slab was often low, yielding of the steel occurred and wide cracks formed. Due to increased fatigue stresses, the durability of the reinforcement and the tendons was no longer assured. Thus the necessity for repair arose. In the late seventies Rostasy and his co-workers [12] developed a technique to strengthen such joints with bonded steel plates. The first successful application was the Sterbecke Bridge near Hagen (Germany) in 1980. In 1986/87 this method was used for the first time with glass fiber reinforced plastic laminates on the Kattenbusch Bridge. The Kattenbusch Bridge is designed as a continuous, multispan box girder with a total length of 478 m. It consists of 9 spans of 45 m and 2 side spans of 36.5 m each. There are 10 working joints. The depth of the twin box girder is 2.70 m. The bottom slab of the girder is 8,50 m wide. One working joint was strengthened with 20 glass fiber reinforced laminates. Each plate is 3200 mm long, 150 mm wide and 30 mm thick. Loading tests performed by Rostasy and co-workers showed a reduction in the crack width of 50% and a decrease of the stress amplitude due to fatigue of 36%. The static and the fatigue behavior was at least equal to the steel plate bonding technique. From the corrosion point of view, the expectations of the glass fiber reinforced plastic laminates are much higher.

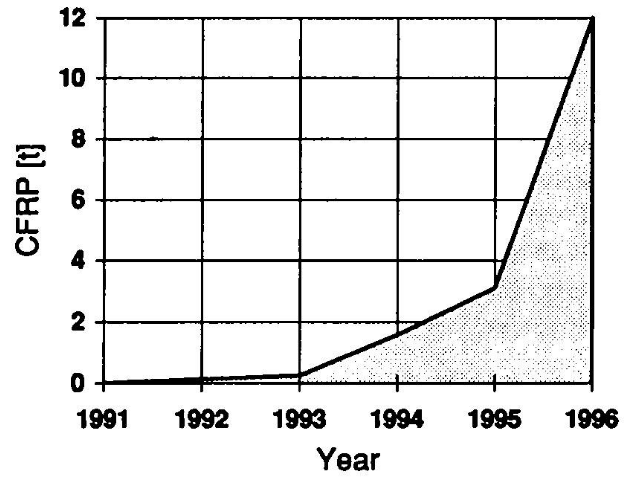


Fig. 2 Use of CFRP-Laminates for strengthening purposes in Switzerland

Another world premiere was the Ibach Bridge in 1991 at the gates of Lucerne in Switzerland. For the first time very thin carbon fiber/epoxy laminates were used to strengthen a bridge. In the following years this method was also used for the historic covered wooden bridge near Sins, the City Hall of Gossau, the large multistory parking garage in Flims, the tall chimney of the nuclear power plant in Leibstadt and the main railway station in Zürich. Beside this projects, which are described elsewhere [13-15] approximately 250 smaller and larger structures were strengthened in Switzerland since 1991 with thin CFRP laminates. From 1991 until 1996 approximately 17'000 kg of CFRP-Laminates were used for strengthening purposes in Switzerland. This mass of CFRP is replacing about 510'000 kg of steel. Figure 2 shows the commercial development. The prices are given in Table 5.

The valuable gothic roof structure of the Church of our Ladies in Meissen in Germany was built in 1447. The gothic vault reaches into the A-shaped cross section of the wooden roof truss. The observed deformations gave evidence that the horizontal tensile members of the "A" did no longer work satisfactory. The masonry of the nave received due to this insufficient action shearing forces from the roof. This fact was underlined by the observed cracks between the vault and the walls of the nave, which opened up to 4 cm. O. Kempe [16] developed prestressed diagonal racetrack CFRP tensile links to relieve the load on the connections of the wooden tensile members and to reduce the horizontal forces of the base points of the "A" produced in filament winding technique. The length of the two types of racetrack links is 1.8 and 3.25 m. The links are connected to the structure, the prestressing elements and to themselves by bolts. Before the successful application and certification of the system trough the German building authorities it was tested in full scale at the EMPA in Dübendorf, Switzerland [17]. In Greece Triantafillou [18] and Schwegler [19] are using CFRP laminates for the rehabilitation of seismic damaged historical buildings in the old part of the city of Patras. Schwegler is using the same method for the same purpose in Zurich, Switzerland [20]. The latest Italian venture is strengthening historical structures with advanced composites. CFRP laminates are used for the retrofitting of masonry vaults, slabs and walls [21].

Table 5: Costs for CFRP laminates including application in Switzerland (cross section: 50 mm by 1 mm; 70 Vol.-% Toray T700 Fibers)

Offer:	Price in US $
CFRP Laminate Grinded On One Side	16.- per m
CFRP Laminate Applied In Easy Going Situations *	85.- per m
CFRP Laminate Applied In Difficult Situations *	120.- per m

*including everything (CFRP-laminate, surface preparation, adhesive, all works, etc. based on Swiss labor costs)

The North Sea Oil Industry upgraded a wind wall to a blast wall on the Mobil operated Beryl B Platform using high strength, high modulus CFRP laminates. This work was part of the safety improvement plan that was a direct result of the Piper disaster and the subsequent legislation requiring safety cases to be prepared for each platform [22].

10. Applications in North America

Several prestressed, adjacent concrete box-beam bridges in the State of Delaware have developed longitudinal cracking on the bottom soffit of the beams. The cause of the cracking was a lack of transverse reinforcement on the bottom face of the precast, prestressed beams, built prior to 1973. Advanced composite materials were used to upgrade such a bridge north of Wilmington. The first in the US bridge rehabilitation field demonstration of carbon fiber tow sheet from Tonen Corp. was successfully conducted in October 1994 by Chajes and coworkers from the University of Delaware [23].

Another important pilot project was the repair of Interstate Highway 95 Bridge over route 702 in West Palm Beach, Florida by Shahawy, Ballinger and coworkers [24] early 1995. A truck traveling Eastbound on route 702 , hit the outermost bridge girder -a 25.9 m long AASHTO Type III prestressed girder. The truck hit caused a longitudinal torsion (twist) in the beam that resulted in two major cracks in the girder. Although the capacity of the girder to carry vehicle loads on the bridge was not substantially reduced, it was necessary to strengthen the beam against possible additional truck hits that could subsequently weaken the girder and the bridge. It was repaired with Mitsubishi Chemical Corporation's Replark carbon fiber sheet. Repair of the damage and strengthening of the prestressed concrete girder involved removal of broken and loose concrete, patching with a repair mortar, installation of the CFRP sheets and protection with a UV barrier paint. The repair was done over a period of a few nights, with a small work crew, with no impact on traffic flow on I-95 and relatively small cost.

11. Applications in Asia

The Japanese are applying carbon fiber laminates for structural strengthening in buildings since the late eighties. In summer 1992 this technique was used for the first time for the retrofitting of a bridge in Tokyo. The method has also here been proven to provide superior external reinforcement performance and potential cost advantages compared with conventional strengthening methods. The Japanese systems are mostly consisting of continuous carbon fiber tows formed into wide sheets adhered to a backing net and removable backing paper are impregnated in the field with epoxy resins. The composite of fiber and resin is adhered to the surface of concrete or masonry to reinforce the structures. The method has found widespread field application. There are over 200 installations to date in Japan alone. The Japanese method is different to the method mostly used in Europe [25]. There pultruded, cured laminates are adhered to the concrete surface. As long as the surface is even there should result no difference between this two methods. If the surface should be uneven, there will be a certain danger that laminates produced insitu follow the contour of the uneven concrete. Therefore exists a certain risk of peeling off the laminate after loading of the structure.

12. Composites For Structural Repair: A Fast Growing Research Field

Among 1992 and 1995 the annual number of publications has been more than tripled. Before 1992 there were not more than five groups working in this field. Today this number is approximately fifty. Especially Canadian teams [26-28] started to be very successful based on their Advanced Composite Materials in Bridges and Structures (ACMBS) Network. The lately announced new Network of Centers of Excellence (NCE) with a total budget of over twenty million Canadian $ will even accelerate Canadian research. It includes the project "Advanced composites and integrated sensing technology for structural rehabilitation". In the USA, Asia and Europe there exists not yet such a good networking like in Canada. In the domains of seismic strengthening of columns and masonry walls there are centers of excellence at the University of California in San Diego [29-30] and at the University of Arizona [31-32]. The Universities of Delaware [33] and South Florida [34] have a lot of competence in flexural strengthening of concrete, steel and wooden girders. The most important Asian contributions are from Japan [25]. In Europe there are research groups in England [35], France [36], Germany [37], Greece [38], Italy [39], Sweden and Switzerland [13-15].

Today's advanced composite materials could solve many of the worldwide rehabilitation problems. They are characterized by the following types of improvement: enhanced durability and service life; superior strength; resistance to corrosion, chemicals, and fatigue; initial and life-cycle cost efficiencies; ease of application; aesthetic and environmental compatibility and ability for structural control. There is an increasing demand for rehabilitation systems with the characterizations given above. Therefore more applied research will be needed. The urgent necessary maintenance work on infrastructure has dramatical implications on the economic life of a nation. Hence government agencies and industries will be ready to sponsor "useful" research. Consequently it will be more and more important that research projects do not end up only with papers and reports but have full scale demonstration projects. That is the most successful way to transfer knowledge from the universities to practice.

13. Concluding remarks

The future is advanced composites. This materials revolution predicted during the 1960s to take place by the turn of the century has, as expected, been more an evolution than a revolution. Confidence is very difficult to build in advanced composites in civil engineering but easily destroyed. Therefore it is essential to use pilot projects to be capable to learn from mistakes and to convince the owners of structures and the building authorities of the outstanding opportunities of composites for structural repair and retrofitting. Up to date there are over 700 field applications worldwide and there are not yet any failures known. The most important factor to remember is not the cost per kg of advanced composite materials, but rather the cost effectiveness of the rehabilitation of a structure, considering the life expectancy and the costs of the alternatives. Figure 3 gives a prediction of the future demand of CFRP for the external flexural post-strengthening with pultruded laminates.

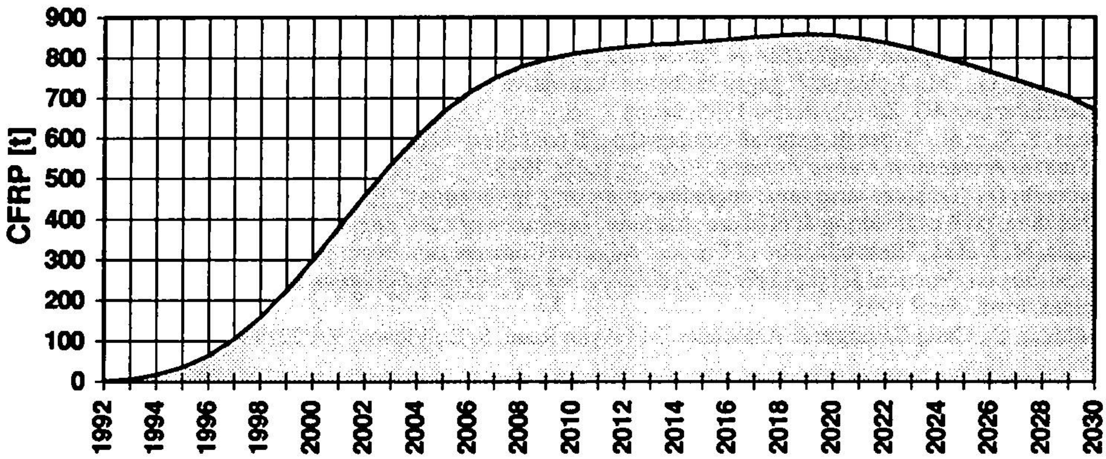


Fig. 3 Prediction of the worldwide demand of CFRP for external flexural post-strengthening

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