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SESSION 1

GENERAL DURABILITY ASPECTS FROM PRELIMINARY DESIGN TO DEMOLITION

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KEYNOTE LECTURES

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Durability of Highway Bridges

Durabilité des ponts-routes

Dauerhaftigkeit der Strassenbrücken

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SUMMARY

Every Bridge Engineer must plan, design and construct bridges, as if they were to safely service the traveling public forever. For only then can the bridge engineer feel that he has satisfactorily executed his commission. He must produce the best possible bridge and the lowest possible cost without sacrificing safety, quality or aesthetics. When he plans, designs and constructs with durability in mind his bridge will become functionally obsolete long before it becomes structurally deficient.

RÉSUMÉ

Chaque ingénieur des ponts doit concevoir, calculer et construire des ponts, comme si ces derniers devaient remplir leur fonction avec sécurité et pour l'éternité. Alors seulement, l'ingénieur a le sentiment d'avoir correctement accompli sa tâche. Il doit construire le meilleur pont pour le coût le plus bas sans sacrifier la sécurité, la qualité et l'estéthique. Lorsqu'il conçoit, calcule et construit un pont en ayant la durabilité présente à l'esprit, celui-ci sera obsolète longtemps avant de présenter des déficiences structurales.

ZUSAMMENFASSUNG

Jeder Brückeningenieur muss Brücken planen, bemessen und bauen, wie wenn diese dem Verkehr für ewig sicher dienen würden. Nur dann hat er das Gefühl, seinen Auftrag zufriedenstellend ausgeführt zu haben. Er hat die bestmögliche Brücke mit den kleinstmöglichen Kosten zu bauen, ohne Kompromisse zulasten von Sicherheit, Qualität und Aesthetik. Plant, bemisst und baut er jedoch schon mit der Dauerhaftigkeit als Ziel, so wird seine Brücke nicht mehr gebraucht, bevor ihre Lebensdauer erreicht ist.

1.0 INTRODUCTION

How long is a highway bridge supposed to last? The obvious answer is that it is supposed to last as long as it is needed. Predicting how long a bridge must remain in service is essential to bridge planning, design, construction, and maintenance. And yet predicting the exact service life of a particular bridge at a given site is impossible. Too many factors are beyond the bridge engineer's control.

Often, the useful life of a bridge ends when it becomes functionally obsolete. The bridge is in good condition, but it is no longer able to carry the traffic loads or volumes existing at that location. This can happen for many reasons. For example, if legal load limits change, or if vehicles are heavier than expected, or if traffic volumes increase as development occurs--the life span of a structurally sound bridge may be shortened. To cite another example, water flow through the bridge opening or the frequency of flooding may have increased to a point that is no longer acceptable. In that case, a new bridge is required to reduce the impacts on surrounding developments.

In the United States, 42 percent of our 577,710 structures are deficient. Of these, 102,531 (18 percent) are deficient only because changes at their geographical location have made them functionally obsolete. In urban locations and high growth areas, more bridges are replaced because of functional concerns than structural considerations.

Of course, some bridges deteriorate to the point where they can no longer carry the necessary loads safely. In the United States, 135,826 (24 percent) of all bridges on public roads are structurally deficient. These bridges have deteriorated to the point that major rehabilitation or complete replacement is necessary.

Ideally, no bridge would be structurally deficient. Every bridge rehabilitation or replacement project would be the result of functional obsolescence, not deterioration of our engineered product. We know how to correct deterioration before it reaches the point where a bridge can no longer serve the motoring public safely. With proper planning, design, construction, and maintenance, this should be possible.

So why have these 135,826 bridges deteriorated? The overall reason is that we do not live in an ideal world. In an ideal world, bridges would be given the attention they need to serve us well for many decades, even centuries. In the real world, we do not always have the luxury to make the right choices. Our predictions about traffic and loadings may prove to be incorrect so a bridge deteriorates faster than expected. Government agencies may not have enough money for all needed maintenance and rehabilitation projects. In the real world, maintenance is deferred, rehabilitation is put off, and a bridge that could have had an extended life of many decades has to be replaced instead. Recent studies by the Federal Highway Administration (FHWA) Bridge Division have shown that the average bridge in the United States is replaced when it is about 70 years old. By then, it would have been rehabilitated once.

Even in the real world, though, this premature deterioration is not inevitable. The cover of the American Society of Civil Engineers' (ASCE) 1989 calendar has a picture of a covered wooden bridge built in the United States in 1866. It is still in service today. For September, the calendar used a picture of an Iron Bridge in Shropshire, England. This bridge, still in use, was built in 1779. These examples and many others prove that we can build durable bridges, if they are properly engineered, constructed, and maintained.

To achieve this durability, we have to explore what can go wrong and, more importantly, what can go right. We cannot control all the factors, such as geographic changes, budget limitations, or traffic volumes or weight. But to a greater extent, we can control the designs we use so they are capable of achieving maximum service life. We can also control the construction and maintenance procedures that will extend a bridge's life cost-effectively. In short, we, as engineers, should be able to select the best type of bridge for a given location, then combine proper materials, quality design, and pride in construction with protective strategies, including regular maintenance, to prevent premature deterioration, be it a timber, concrete, or steel bridge.

If we look at each stage in the life of a bridge we can find ways of increasing durability to extend effective service life. The first stage is planning.

2.0 PLANNING

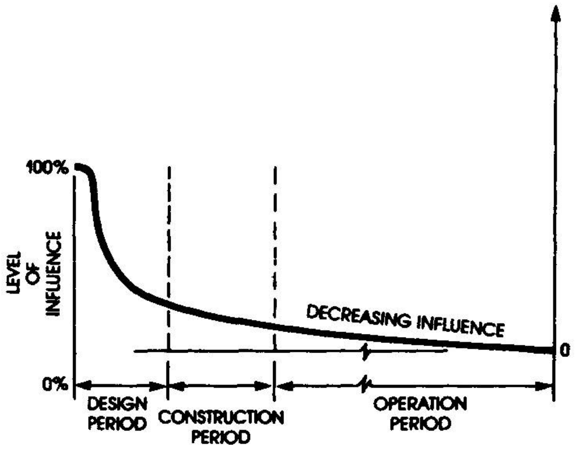
During the planning phase of project development, engineers determine the type, size, and location of the bridge. Site or other constraints dictate size and location. The type of bridge, however, is usually open, and the choice can have a significant effect on durability. We can see how by using an extreme example. We would not select a bridge for an Interstate highway that could not serve high volumes of traffic, much of it heavy trucks, well into the next century. True, some bridges may be the lowest first-cost solution. But that ignores the cost of replacements every 20 or 25 years and, more importantly, the cost of delays and inconvenience of traffic disruption each time. Therefore, in the planning stage, we would program a bridge-type with a higher initial cost, but one that incorporates more "durable" materials.

The planning phase is also when the impact of the environment on the bridge is first discussed. To again cite contrasts, we can see that a bridge carrying mostly automobiles over a slow-moving river in a warm climate should be different from a bridge carrying a traffic with a high percentage of trucks over ocean salt-water in a cold climate with snow and ice throughout the winter. While this example suggests the difference between Florida and Alaska, even bridges within an urban area, or a county, or State can be subjected to considerably different environments--for example, piers on solid ground or in the bed of a rushing river. During the planning stage, bridges must be programmed to incorporate materials suitable to the environment.

If these factors are not recognized during planning, and adequate funds are not made available for design and construction with durable materials, the resulting bridge will not last as long as it could have.

3.0 DESIGN

The design phase has the greatest impact on the quality--another name for durability--and cost of the bridge. In a publication entitled Quality in the Constructed Project, {1} the ASCE included a chart that illustrated the effect on quality: (Figure 1).



Opportunity to Influence Project Quality and Cost

Figure 1

The ASCE defines quality as meeting the needs of the owner within the available budget. For some bridges, this definition may appear to consist of two mutually exclusive terms. The owner may want a bridge that will last 100 years but may have a budget that appears to cover only a 50-year bridge. That is why experienced designers play such an important role. The designer must know how the various engineered materials, under the given site conditions, will perform. That way, the designer can get the most value, if not 100 years, for the available funds. To borrow an old definition of "politics," bridge design is the art of the possible.

Bridge engineers decide on materials during the design stage. The two broad choices are usually concrete or steel. Within the concrete and steel industries, the pros and cons of each type are controversial. From the standpoint of durability, though, the type of bridge is less important in most cases than the choices made after the type is chosen. What the designer has to recognize is that each setting is different. The skill of the engineer comes in understanding the differences, and making choices accordingly. We will illustrate this point with a few examples, beginning with concrete.

Many engineers include high-strength concrete in their designs. These engineers think that high-strength concrete is more durable than lower-strength concrete. That sounds logical enough. And if stresses and strains were the only consideration, this assumption would be true. But the environment shows no respect for high-strength concrete, if such ingredients as "entrained air" are not included. Air entrainment reduces the strength {6} of the mix, if all other ingredients are kept constant (cement, water, etc.), but revising the mix design can provide both the needed strength and air entrainment.

Another factor that makes a major contribution to the durability of concrete bridges is the permeability of the final concrete mix. The lowest water-cement (w/c) ratio, compatible with workability, has to be specified for exposed concrete elements. Concrete with a w/c ratio of 0.5 (5 1/2 gallons per 94-pound bag of cement) has a higher permeability than concrete with a w/c of 0.4. The lowest practical w/c that can be placed is about 0.32 (depending on temperature), and then only with special equipment for consolidation. To get a durable concrete, therefore, the designer has to recognize the interrelationship of strength, permeability, and workability when specifying the concrete mix required for the project. The designer should also know that adding high-range water reducers, fly ash, retarders, silica fume, etc., may require a different w/c ratio to achieve equivalent quality levels.

As with concrete, the durability of structural steel is influenced by the decisions made during the design phase. To cite one example, designers who do not keep up with the latest in fatigue resistant design are potentially burdening their client with excessive rehabilitation costs to achieve expected service life. This can occur, for instance, when a structural detail develops a fatigue crack that grows until it becomes a critical flaw. If the structure is non-redundant and the member fractures, the entire structure can be lost. These cracks rarely start because of the design of the main load-carrying members. If proper detailing is not used for the secondary members, however, "out-of-plane bending" that is not accounted for in the design can, and often does, cause cracks.

Too often, designers incorporate fatigue-prone connections in new structures well after the technical literature has fully documented the weakness of the joint. In this case, the designer has made the choice of a bridge-type, namely steel, that can last many decades. Then a secondary choice may well cut that life short, with tragic consequences for any motorists on the bridge if the member cracks.

In making structural material choices, the designer must decide how to account for life-cycle cost in the analysis. Because of the rivalry between concrete and steel, this is always a controversial topic. No one answer seems to satisfy advocates of both types. Still, life-cycle cost is important to the owner of the bridge. Therefore, life-cycle cost must be important to the designer. It is influenced more by the designer's details than basic structural materials.

Life-cycle cost is the cost of all activities associated with a bridge during its life. It includes the cost of construction, but also the cost of any maintenance, rehabilitation, or reconstruction that may be needed during the life of the structure. Life-cycle cost allows designers to compare the cost of alternatives over the life of a bridge, instead of simply initial cost. A bridge with a low initial cost may be within a bridge owner's construction budget, but could be a poor investment if it requires rehabilitation three times during its service life instead of two.

In looking at the structures in the United States, we can see that any type of structure, properly designed, constructed, and maintained will have, comparatively, the same life-cycle costs. Therefore, the bridge owner's goal must be to develop a process that will allow only durable bridges to evolve.

Durability, in this regard, should be defined as a combination of engineered products or materials that will satisfy project needs at a specific site for a specific design life. This definition is useful because it makes clear the fact that "durable" does not always mean a "long-lasting" structure. Even though we know a concrete bridge, for example, could last 50 years or more, if the completed bridge will be replaced in 10 years because of functional obsolescence, concrete might be a poor choice.

In considering life-cycle cost, we sometimes find that the calculations are less persuasive than the experience of the designer. Should the steel be painted or unpainted (so-called "weathering" steel)? Should the concrete be reinforced or prestressed? Depending on who is involved, different selections will be made based on experience. An engineer who has dealt mainly with steel structures during his professional life would probably argue the advantages of steel in life-cycle cost. Engineers who have dealt mostly with concrete structures would probably think the facts favor concrete.

Some aspects of life-cycle cost, though, have been recognized. Concrete bridge decks are an example. Premature deterioration of bridge decks--deterioration occurring before the end of the bridge's service life--has occurred in - localities where deicing salts are used to meet the need for winter travel. Because of the recognized effect of salt on reinforcing steel in concrete decks, experienced designers incorporate a "corrosion protection" system with higher quality concrete. That combats the influence of the salt, but with a higher initial cost. By calculating life-cycle cost, we can see that this extra cost is justified because the deck will now have a service life equal to the rest of the structure.

We can see a similar debate over the life-cycle cost of painting. A structural steel member, if properly designed, detailed, maintained, and painted will last, in theory, forever. The 1779 Iron Bridge mentioned earlier is an example. But repainting steel bridges can be costly and it affects traffic, presenting safety hazards to workers and motorists. These costs have to be added to the life-cycle analysis.

Many designers think the answer to the painting problem is "weathering" or unpainted structural steel. That avoids the added cost of repainting. Moreover, used in the proper environment with appropriate details, "weathering" steel will provide an acceptable service life with minimal costs. "Weathering" steel, though, is not durable in all environments. In a marine environment, "weathering" steel experiences accelerated, premature deterioration, with resultant maintenance or rehabilitation costs for the owner. Because of windborne salts, this deterioration may occur even though the structure is miles from a seacoast. It may occur hundreds of miles from a marine environment, in fact, if roadway deicing salts come into contact with the steel. The designer must have a full understanding of the limitations of this material in calculating life-cycle costs.

Bridge joints are perhaps the single biggest cause of premature deterioration of bridge components. In addition, they greatly influence the rideability of the roadway surface. Here, too, the designer must make choices that will affect the durability of the bridge as well as its life-cycle cost.

One such choice involves handling water that passes through the joint. Because water is often laden with salt, it could cause the superstructure and the substructure to deteriorate. Experience has shown that joints will not remain watertight over the service life of the structure. Aging joints, by leaking, can affect durability. At the same time, replacing joints every 5 to 10 years to retain water-tightness is unacceptable, especially on high traffic routes. To meet these circumstances, a skilled engineer may consider provisions to control the water under the joint rather than try to prevent the water from penetrating. Properly detailed and maintained joints can control this water for the life of the structure.

For smaller structures, "jointless" bridges have been used with great success in some States. Tennessee, for example, {8} has built 400 foot long steel, and 800 foot long concrete bridges without joints. The State has not had any significant problems as a result of this design decision. Of the 577,710 structures in the United States, over 90 percent are less than 500 feet long and may qualify for a jointless design. Even though this design could increase durability, designers use it infrequently.

Bridge drains are another area where water flowing out of control may subject a structure to significant damage. They are used too frequently on highway bridges, probably because of the thinking that "more is better." In fact, the reverse may be true. With fewer drains, more water must flow through each one, thus "flushing" out the drains. Maybe the answer to many of our bridge problems is "jointless and scupperless bridges." An FHWA research report on Bridge Deck Drainage Guidelines {2} provides good advice for the engineer on deck drainage design.

The number of drains is an example of how a little decision can create a big problem. We do not want to slight the big decisions. A designer should not develop a structure where premature deterioration of any single element, from whatever cause, will require total replacement of the bridge. This is the basic idea of redundancy. If one thing fails, another will do the job. Designing for redundancy may increase initial project cost by a small percentage. That small increase may affect the competitiveness of the design in comparison with another structural system. Nevertheless, every designer and every bridge owner has to recognize the importance of redundancy. We are not comparing apples-to-apples if we try to decide whether to use a bridge design that can survive the failure of a critical member or one that cannot.

If the structure will cross a river or stream, the designer has several other choices to make that, obviously, can affect durability. A bridge must be able to withstand major floods. Many bridges cannot, though. More structures are lost in the United States because of floods than for any other reason. We may blame these losses on "acts of nature," and that is correct up to a point. But the losses often are a direct result of designer and owner decisions.

Scour resistant designs, for example, should be mandatory for structures that cross a body of water. If the bridge footing cannot be placed on a non-erodible base, such as competent rock, the designer must find ways to build in stability. The bridge must be able to withstand the "design flood" (usually a 100-year storm) without damage. In addition, it must be able to remain stable at even greater flood frequencies (say a 500-year event), although with a smaller factor of safety.

Because initial cost is, inevitably, a consideration, this stability must be achieved without increasing the cost exorbitantly under the guise of safety. To a degree, this is risk management. How much additional cost will the owner be willing to bear so the structure can withstand a 100-year flood, a 200-year flood, and so on.

A well-engineered foundation should be able to withstand a "design flood" or worse without any significant changes in cost or constructibility. If the designer makes all the other best choices, but does not give proper weight to scour resistance, he will have saddled the bridge owner with a structure that is not as durable as it could have been.

In 1988, the FHWA issued a Technical Advisory {7} providing guidance on scour resistant designs. Using the procedures described in the advisory will result in scour-resistant, and thus durable, bridges.

4.0 CONSTRUCTION

The best planning and design can go for naught during the construction stage--if the contractor is not "quality conscious," if the inspectors are not conscientious, if any one of a thousand details are not done as shown in the plans, the bridge will not be as durable as it should have been.

In some cases, durability and quality may be compromised, even before construction begins, by the procedure used in selecting contractors. To secure the benefits of cost competition, bridge owners select contractors by "low-bid." To stay in business, each bidder must minimize time and cost for the project, yet ensure the bid price will provide a product that meets the contract requirements. Bidders will not adopt a more costly or more time-consuming approach--they will stick to the specifications, no more and, we hope, no less. Thus, it is extremely important that the owner specify the type of structure and appropriate material specifications and procedures that have proven to be durable in other similar circumstances.

To be sure minimum quality level standards are met, the bidding documents must clearly indicate the requirements for the project. The criteria that will be used for acceptance or rejection must be clearly spelled out. Further, a balance must be struck. The contract documents should not over-specify material requirements or impose unnecessary construction restraints. And yet they must not be so ambiguous that the contractor will submit a bid thinking a lower priced product will be acceptable.

The bidder's level of experience is a consideration that is often overlooked, especially for state-of-the-art structures. The bridge owner and the contractor should be more conscious of experience. Many highway departments require financial prequalification before a contractor submits a bid, but few require technical qualification as well. The lack of technical prequalification has caused many problems when inexperienced contractors have used the claims process to try to recoup losses resulting from their not fully understanding the complexity of the work they bid.

Some bridge owners have begun requiring technical prequalification to minimize these problems. Industry, too, has recognized the need for technical prequalification. For example, the American Institute of Steel Construction has a three-level certification program for steel fabrication. Two of the levels deal with fabrication of steel bridge members. The Prestressed Concrete Institute has a four-level bridge member certification program. These programs require contractors (or precasters) to satisfy minimum quality control standards to become certified for the appropriate type of construction. Technical prequalification, combined with an adequate design and adequate contract provisions, could eliminate some of the problems we are seeing today.

Quality control is the responsibility of the contractor. Acceptance testing is the testing performed by the owner's representatives to be sure the contract provisions are met, and a durable bridge will result. Together these form the concept called Quality Assurance (Figure 2). The contractor must have an adequate quality control program. Just as importantly, the owner's representatives must understand the acceptance testing criteria and how to interpret the results to be sure the project will result in a durable structure meeting contract requirements.

Curing of freshly placed concrete offers a good illustration of the importance of quality construction. It is perhaps the least understood and most abused stage of concrete construction. This is especially true for bridge decks,

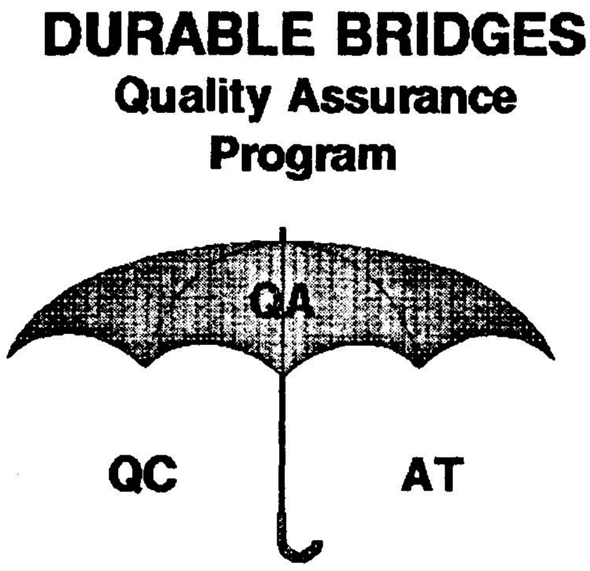


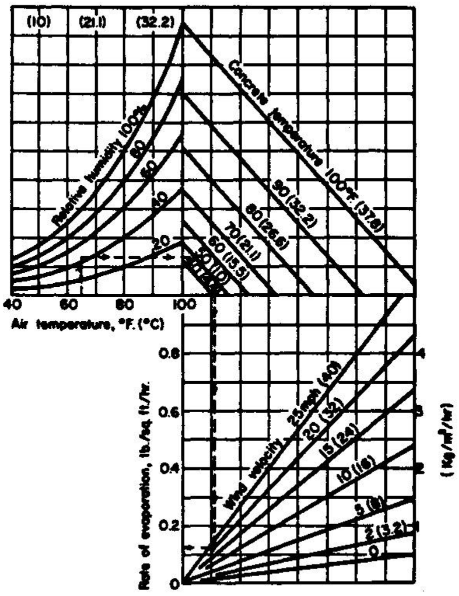
Figure 2

where large surface areas are exposed to ambient conditions. Timing is vital to curing, but proper application is also important to ensure controlled evaporation rate of hydrating water. Otherwise a less durable, poor quality product will result, with early bridge deck failures occurring even if everything else during the design and construction phase was done properly.

The Portland Cement Association (PCA) addressed curing in its manual on *Design and Control of Concrete Mixtures* {3}. Concrete with exposed surfaces should not be placed if the evaporation rate exceeds, or is expected to exceed, a rate of 0.25 lbs/sf/hr. Figure 3 shows the PCA chart for calculating this rate. It considers humidity, wind velocity, and the temperature of the air and the concrete. Will the contractor measure these factors? Will the inspector be sure the measurements are taken? Will the construction manager catch this detail? If not, the results will be less durable concrete than the designer expected. Proper curing cannot make bad concrete good, but bad curing can make otherwise good concrete bad.

5.0 MAINTENANCE

Proper maintenance does not happen often enough. Because of reduced budgets or inadequately trained personnel, many organizations delay needed maintenance. Deferred maintenance that could have been performed at minimal cost leads instead to rehabilitation at major cost. All too often, we use our limited funds to rehabilitate a bridge that should not have needed rehabilitation for many years, and we replace a bridge that should have lasted many more years, all because of deferred--or ignored--maintenance. Deferring maintenance, therefore, is not only a false economy, but one that can directly affect durability.



Effect of concrete and air temperatures, relative humidity, and wind velocity on rate of evaporation of surface moisture from concrete.

Figure 3

The bridge designer, in considering life-cycle cost, should be able to tell the bridge owner the type of maintenance a bridge will need. The bridge owner should use this information to establish a maintenance program for the bridge before it is opened to traffic. Having the program is not enough, though. The bridge owner must stick to the schedule. This schedule should be like the maintenance schedule that comes with every new car--the owner ignores it at his own risk.

Something as basic as repainting bridges is often deferred. When painting is deferred, for whatever reason, the bridge owner finds that what should have been periodic simple surface preparation with a new top coat is not sufficient. That simple job must be replaced by extensive blast cleaning, priming, and top coating. That is how a forced economy for lack of funds becomes a false economy, whether funds are available or not. In addition, while "saving" money on repainting, the bridge owner is exposing the bridge to the harmful influences of the environment that can shorten the life of the bridge.

Often, funding for regular repainting is not available. Bridge owners, however, usually overlook the next-best possibility. Spot painting can help to protect the more vulnerable areas of the bridge, such as under joints, and to replace sections of paint that failed prematurely. This maintenance strategy is rarely used, but it can significantly reduce the cost of retaining durability.

As in the contracting field, certification of painters can make a major difference in the quality of work. In the United States, the Structural Steel Painting Council is establishing a certification program for painters to be sure minimum quality levels are achieved. Bridge owners who do not rely on the certification can only hope the low bidder selected for the job is capable of at least minimum quality.

"Weathering" or unpainted steel, mentioned earlier in the discussion of life-cycle costs, eliminates one maintenance problem, namely regular painting. However, bridges built with "weathering" steel need maintenance, too. Debris that collects on bridge members from pigeon nests, leaking expansion joints, wind blown dust, or other sources must be removed promptly. This material retains moisture and will cause accelerated corrosion. Salt-laden water coming through bridge joints evaporates, leaving salts that are highly corrosive. They must be flushed off. Some bridge owners have adopted a maintenance policy of washing their bridges, both painted and unpainted, at least once a year to extend their service life. This simple, basic practice is sure to pay high dividends in durability.

"Weathering" steel offers a good example of how a choice made during the design stage affects later stages. The designer may choose this type of steel to reduce maintenance costs and avoid traffic disruption. But that choice imposes special maintenance requirements on the bridge owner. If the bridge owner meets those requirements, the bridge will provide good service at minimal cost. If the bridge owner does not, the choice of "weathering" steel will lead to needless rehabilitation costs and possible replacement of the structure years before its useful life should have run out.

Concrete surfaces also require attention. Debris, for example, should not be allowed to build up and remain on pier caps or abutment seats. Something as simple as the failure to clear debris from drainage openings can prevent a carefully designed, carefully constructed drainage system from operating properly. Instead of moving through the proper channels, polluted, or salt-laden water may sit on the bridge for long periods or may drip over the side.

For surfaces under bridge joints, concrete sealers are recommended to provide the needed service life. In the United States, we often consult the Transportation Research Board's Concrete Sealers for Protection of Bridge Structures {4}. It provides the results of tests of numerous commercially available products that will seal the concrete surface and provide adequate protection.

We have already discussed the corrosive effect of deicing salts on bridge decks and some of the steps that can be taken to minimize this effect. Maintenance of bridge decks, however, may be needed for other reasons as well. The riding surface may rut or its skid resistance may be reduced by wear from high traffic volumes. Restoration of these properties is necessary under the maintenance program to ensure a safe, smooth riding surface on the bridge.

An asphalt overlay is the least costly of many ways of correcting the problem. If the concrete deck has an {3} adequate air entrainment system (as determined by ASTM Test Method C457) and a corrosion protection system, asphalt may be all that is needed. If either of these two characteristics is not present, an impervious overlay (concrete or polymer concrete) or membrane may be needed. The American Association of State Highway and Transportation Officials, the Associated General Contractors of America, and the American Road and Transportation Builders Association have set up a joint committee to develop comprehensive guidelines for overlays. The results will be published soon as a guide specification.

Another feature that inexperienced designers often overlook is "maintainability." For example, access to vulnerable parts of the structure must be convenient enough to allow effective maintenance. Access holes into closed spaces (box sections) must be large enough to allow easy access for personnel as well as equipment. It makes no sense to have to shut down traffic and remove a portion of a bridge deck to get inside a box girder for maintenance.

"Inspectability" is of equal importance. The designer must allow for easy inspection of the entire structure. This helps to ensure early detection of problems. Discovering a fatigue crack when it is 2 inches long allows easy, economical repair. Compare that to shutting down the bridge and installing falsework to repair a fractured element because the inspector could not inspect a particular detail. The designer should mentally "inspect" the bridge while designing it to ensure inspectability. That way, to cite another example, if he or she has a 36-inch girth, he or she won't detail 24-inch access openings.

6.0 TOTAL REPLACEMENT

At some point in the life of a bridge, it may no longer be cost-effective to continue maintenance. This can happen for many reasons. For example, perhaps the rate of deterioration is so great that it cannot be coped with. Or perhaps traffic volumes or other environmental factors have changed significantly, making the bridge functionally obsolete.

Deciding whether to replace a bridge or not is difficult. Cost, of course, is one reason. Another reason is that in most cases, the structure is carrying highway traffic that will have to be detoured during the replacement project. During the planning, design, and construction stages of a replacement bridge project, the same kinds of decisions must be made as in developing a bridge on new location. However, additional factors must be taken into consideration. For this type of work, the decisions made after the type of bridge is chosen may be even more important.

For example, the designer may choose higher cost materials such as polymer concrete instead of portland cement concrete. The higher cost materials can achieve higher strength in shorter time. To meet the project's time needs, precast elements may be favored over cast-in-place elements.

Contractors and engineers versed in the latest techniques for design and construction while maintaining traffic can, and have, produced fully satisfactory, durable bridges. Often, the bridge can be constructed in a remarkably short time. Because time is so important in these cases, bridge owners have offered significant incentives (namely bonuses) for early completion and penalties for late completion. These incentive/ disincentives have proven successful. They can help get the new bridge open faster than would otherwise be the case--and without sacrificing durability in the name of early completion.

Many times, only partial rehabilitation, such as bridge deck replacement, is needed. The question that has to be answered is what is the remaining life of the rest of the bridge? Will the beams, for example, last as long as the planned deck? If not, is the deck "over-designed" and, therefore, a poor investment?. In 1988, the Bridge Subcommittee of the American Association of State Highway and Transportation Officials adopted a guide specifications entitled <u>Guidelines for Strength Evaluation of Existing Steel and Concrete Bridges</u> {5}. Considerable professional judgment, coupled with these guidelines, will allow the proper decisions to be made.

7.0 CONCLUSION

We would like to able to say that all of the above items--and many we did not have time to mention--are accounted for in each and every project. But history has shown it has not happened. With the diminishing resources available to bridge owners (time, money, and trained staff) and the large number of bridges still in the "deficient" category, strict attention must be paid to the need for quality planning, design, construction, and maintenance. These project phases should be incorporated into a comprehensive Bridge Management System to be sure available resources are used effectively.

In closing, and in the words of English author, art critic, and social reformer John Ruskin:

> Therefore when we build, let us think that we build forever. Let it not be for present delight, nor for present use alone. Let it be such work as our descendants will thank us for, and let us think, as we lay stone upon stone, that a time is to come when those stones will be held sacred because our hands have touched them, and that man will say as they look upon the labor and wrought substance of them, "See, this our fathers did for us."

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General Considerations on Bridge Durability

Considérations générales sur la durabilité des ponts

Allgemeine Betrachtung über die Dauerhaftigkeit von Brücken

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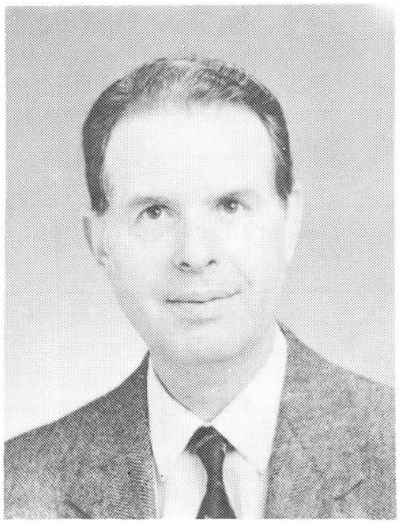

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SUMMARY

The life expectancy of bridges is somewhat variable, but economical considerations require it should be high, at least at the scale of a century. This may imply some changes in practice concerning design and contracting procedures, in order to improve quality, a condition of durability. Furthermore, the recent efforts of many countries in favour of maintenance must be further developed.

RÉSUMÉ

La durée de vie possible des ponts est assez variable, mais des considérations économiques exigent qu'elle soit élevée, au moins à l'échelle du siècle. Ceci peut conduire à des changements d'habitudes dans les procédures d'étude et de passation des marchés, en vue de promouvoir la qualité, condition de durabilité. En outre les récents efforts de nombreux pays en faveur de la maintenance devront encore être développés.

ZUSAMMENFASSUNG

Die Lebenserwartung von Brücken variiert von Fall zu Fall, sollte jedoch aus wirtschaftlichen Gründen in der Grössenordnung von 100 Jahren liegen. Diese Forderung bedingt gewisse Aenderungen der Projektierungs- und Ausschreibungsmethoden, um die Qualität, eine Grundbedingun der Dauerhaftigkeit, zu verbessern. Darüber hinaus sind die Anstrengungen zahlreicher Länder für einen besseren Unterhalt weiterzuführen.

1. ECONOMICAL CONSIDERATIONS

For bridges the concept of lifetime is of great importance, as otherwise the demands placed on public maintenance repair and replacement budgets would be too great. Bridge construction is an ancient activity of civilized societies ; as nowadays many old bridges are still suitable for service, we have been thinking for a long time that long lifetimes could be expected from bridges. Nevertheless we must admit today that the efforts of modern technology have aimed more to lower the cost of structures than to improve their durability.

An interesting report of the OECD, entitled Bridge Maintenance and dated 1981, gives some data, which we reproduce below, concerning the annual rate of bridge replacement in different countries.

Country	Rate of Bridge Replacement	Rate % per ann.
Belgium	No data. As a first approximation assume a life of 100 years	
Denmark	2-4 bridges per 1,000 at present.	0.2 to 0.4
Finland	18 bridges per 1,000 and 10 culverts per 1,000 during 1978.	1.8
France	142 bridges per year out of approx.50,000 (average for 1976, 77, and 78)	0.3
Germany	Overall replacement rate	0.6
Italy	5 motorway bridges out of 1,200 replaced during last 20 years	0.02(*)
Netherlands	1 bridge per 1,000 (due to technical obsolescence), on the State Highway System, which started in the 1930s	0.1(*)
Norway	16 bridges per 1,000 on National Roads (average of 1977 and 1978)	1.6
Sweden	Estimated at 6 bridges per 1,000	0.6
United Kingdom	Estimated at 4 bridges per 1,000	0.4
United States	3,620 out of 258,000 Federal Aid bridges being replaced over a period of 7 years.	0.2

(*) These rates relate to systems containing a high proportion of new structures

The reasons for replacement were not recorded, but some countries indicate that their rate was limited by the funds available. If we do not take into account the low rates, which concern highway system containing a large proportion of new bridges, neither the high rate, due to replacements necessitated by a change in vehicle regulations, the range appears to be between 0.2 per cent and 0.6 per cent per annum, with an exceptionnal high rate of 1.6 per annum in Norway.

A rate of 0.6 leads to an assumed serviceable life of 170 years, and the rate of 0.2 raises this assumed life up to 500 years, what is obviously far too much. But this approach takes into account only an average replacement rate, which should be available for long periods. Now in all industrialised countries many more bridges were built in the last few decades than in the past, and this fact tends to lower the average age of the stock of bridges. Three examples of age distribution are shown in Figure 1, concerning the stock of bridges of two German Länder and of the United Kingdom. They show that in these regions 90 per cent of the bridges are less than 50 to 80 years old.

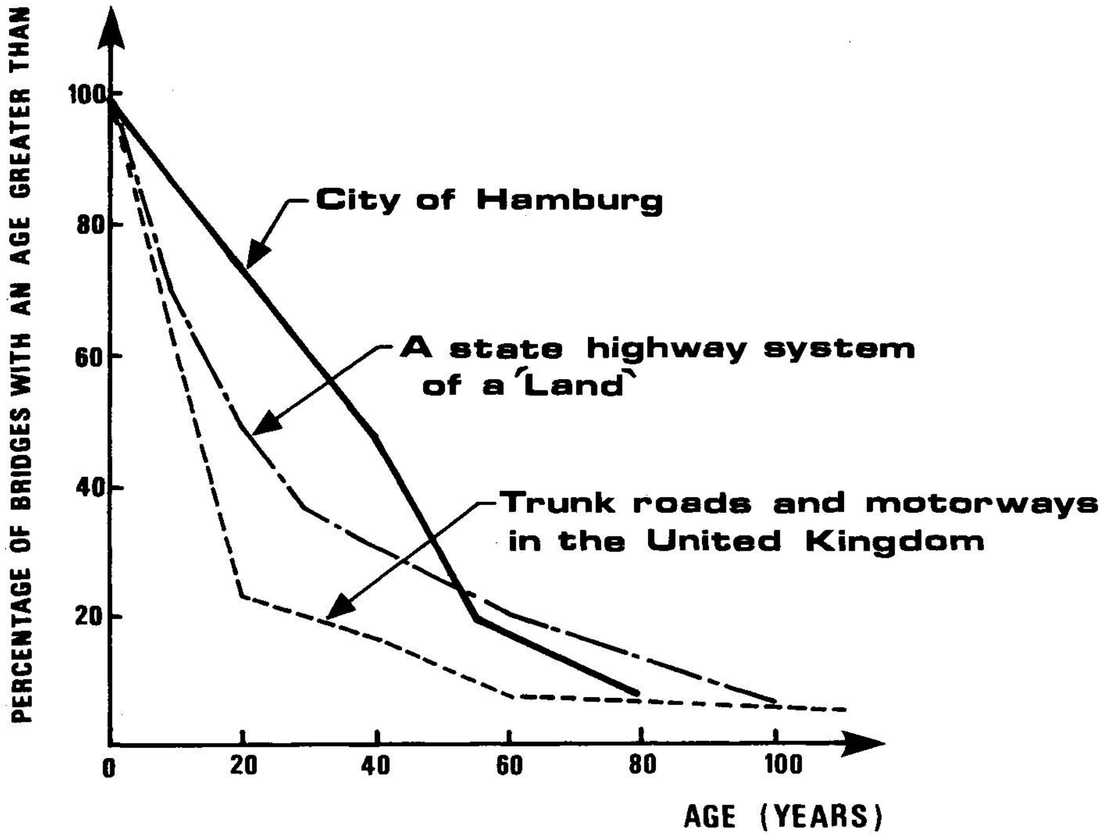

Fig. 1 : Example of age distribution

How it is possible to take into account the age distribution is well illustrated by a study of the German Land of Rhine Palatinate concerning bridge construction planning. The "bridge generation cycle" is assumed to be 60 years, and is shown in the form of a spiral in Figure 2.

Starting from the total number of bridges existing in 1918, this graph shows the number built each year between 1918 and 1947, then between 1947 and 1977. During the period 1977-2007 the bridge building programme will consist on the one hand of new structures (over 1,000), corresponding to the extension of the existing stock, and on the other hand in the replacement of bridges built between 1918 and 1947, when they reach the age of 60 years, the total number of the latter being greater than the former. Then, after 2007, the bridges built between 1947 and 1977 will progressively need replacement.

The method is interesting, although a life expectancy of 60 years seems to be a little short. Moreover, it neglects two factors : first, the deck area to be rebuilt would perhaps be a better parameter than the number of structures, as the latter includes large and small structures. Then, bridges of various ages were built with different techniques, and there is no reason to assume that different techniques lead to the same expected life.

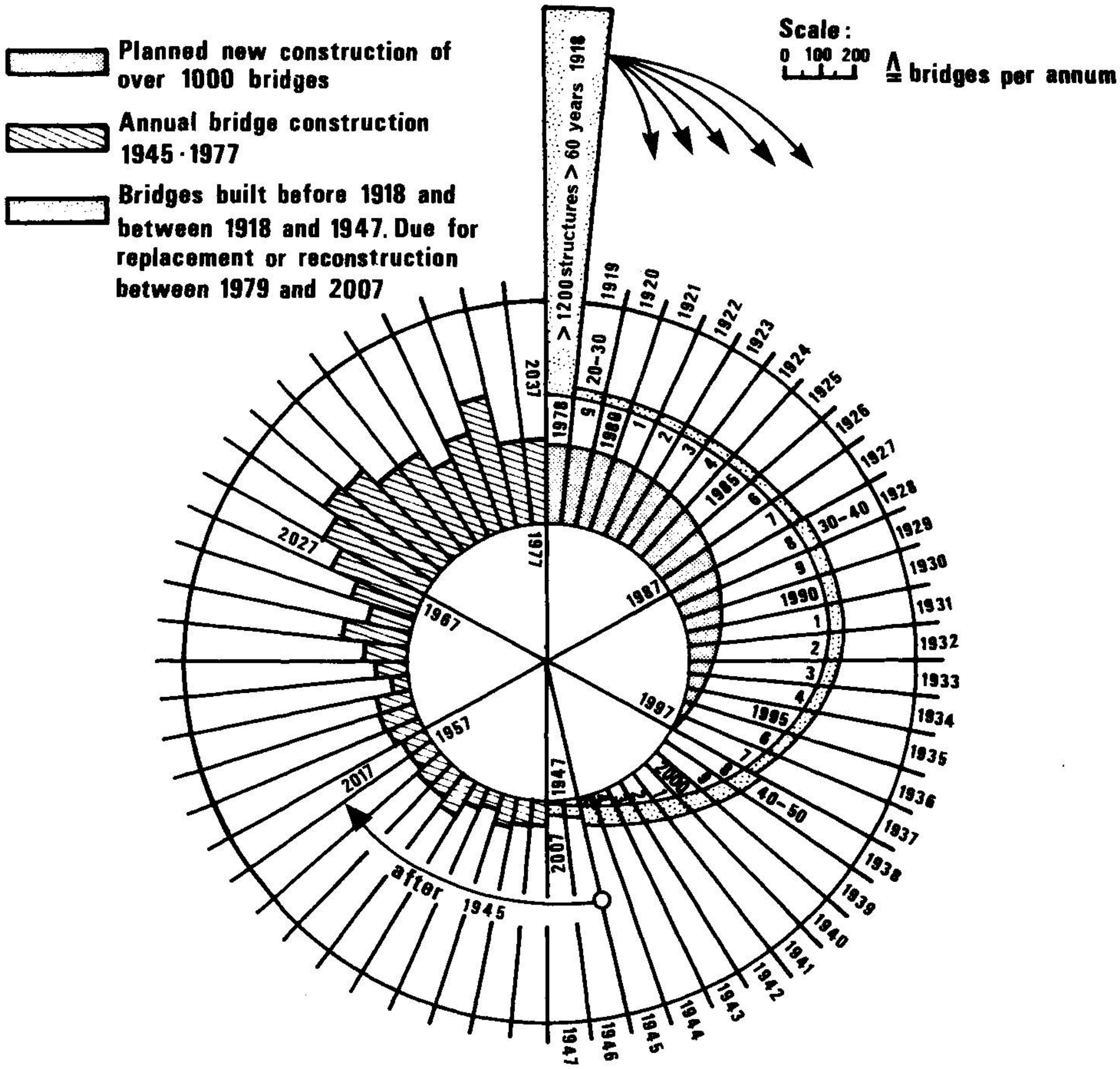

Fig. 2 : Bridge generation cycle of Rhine Palatinate Land

In France, for example, there are no detailed statistics concerning the materials of bridges and their state of maintenance, but a survey of the repair files established for financing purposes as well as the experience of local Départements make it possible to give an idea of the durability differences between techniques.

Many bridges, especially small and medium-sized in the local networks, are still masonry arches. They often have suffered from lack of maintenance, but because of their strength they often present sufficient serviceability, sometimes after some repairs. It also appeared that the foundations of the old masonry bridges on large rivers frequently were not deep enough to escape undermining, as a consequence of the technical limits imposed when they were built. But a foundation strengthening is possible, and if done, puts them in excellent serviceability.

As far as the old steel bridges are concerned, they generally support fairly well their growing old, if the painting has been renewed with sufficient frequency.

They were indeed fairly liberally dimensioned, because of the lack of knowledge in the calculation of structures, and fatigue does not seem yet to affect them. An exception, however, is to be mentioned concerning all steel bridges the deck of which is made out of little masonry arches : the latter are not waterproof and the corrosion has often strongly attacked the steel pieces supporting the deck.

Fig. 3 : Old steel bridge

Another exception, the suspension bridges : there are a little more than 200 of them in France, the major part of them being built a fairly long time ago and located in the secondary networks. Their load carrying capacity is frequently insufficient, and their maintenance condition rather poor. The collapse of one of them during the particularly cold winter of 1985 pointed out the vulnerability of the suspension bars of most of them, under low temperatures, due to the steel quality used at the time.

Fig.4 : The Sully suspension bridge

Further, the reinforced concrete bridges fall into another category. Some of those built in the first decades of reinforced concrete construction are beginning to need replacement. At that time indeed, no means of correct vibration were available, and the concrete density was insufficient. Modern concrete bridges seem to have a good behaviour, but will this last for a long time, particularly with salt aggression : we simply do not know. Many countries have suddenly encountered rather serious corrosion problems with the use of salt on roads during the winter. In France fairly high percentages of cement in the concrete used to be employed, and this

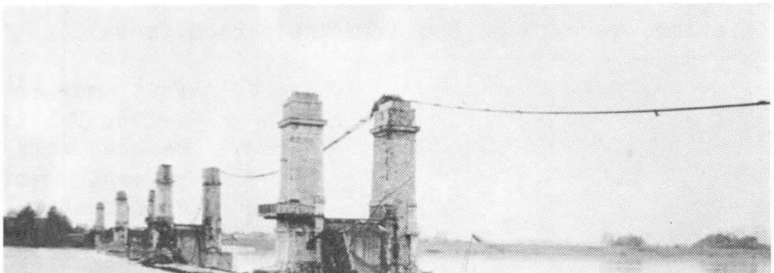

Fig. 5 : The same after collapse on Jan.16. 1985

explains perhaps the now prevailing good condition of the relatively recent concrete bridges. Nevertheless the appearance of some cases of alkali-aggregate reaction in the North of France points out a danger which up to now had not yet appeared in this country, and nobody knows whether this problem will remain limited or not in the future.

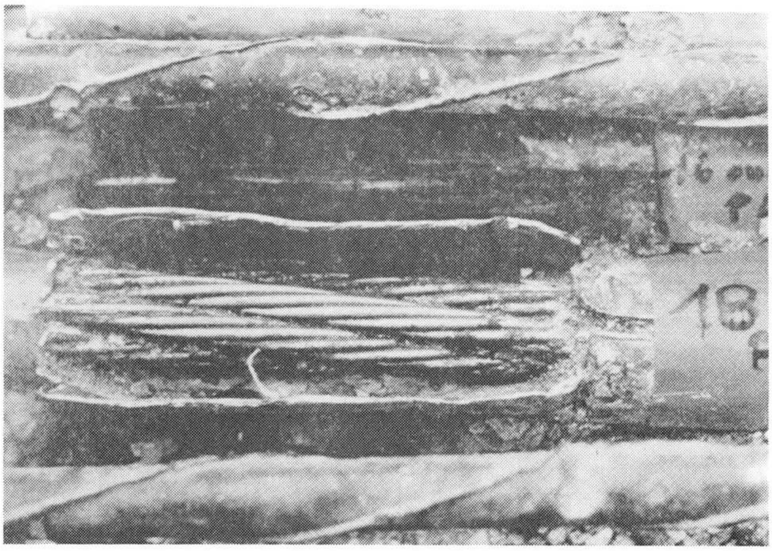

Fig. 6 : Attack of the tendons by corrosion

Finally, the last technique of primary importance to appear in the field of bridge construction is, of course, prestressed concrete. The first structures built in the 1950's were made out of precast in-situ prestressed beams. Many structures of this decade present lack of grouting and insufficient waterproofing, so that the tendons have been severely attacked by corrosion. It has been necessary to replace several structures of this type.

Another defect, which appeared in prestressed box-girders of the 1960's and the beginning of the 1970's is the lack of prestressing, due to some effects now well known. This lead to repairs by additional prestressing, which do not seem to have a noticeable effect on the life expectancy of these bridges.

Another indication concerning the life expectancy of bridges is given by following little statistics concerning 501 bridges replaced in France between 1978 and 1983. 13 of these only, all masonry bridges, were more than 200 years old and in the period from 150 to 200 years 15 were masonry bridges and 3 metal bridges. The highest densities of replaced bridges, according to their actual lifetime, were to be found in the period from 75 to 100 years.

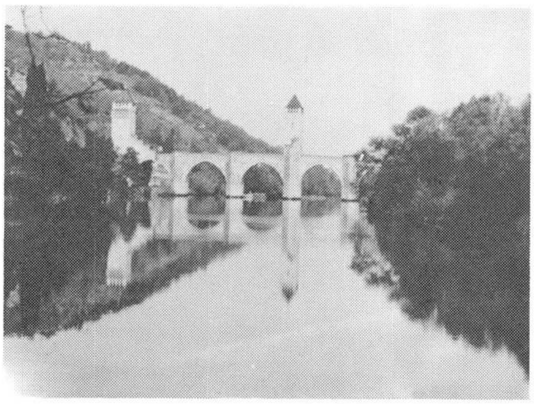

Fig. 7 : Masonry technique reached its maturity through centuries

What may be concluded from this brief survey ? The old technique of masonry reached its level of maturity through several centuries and it left some comfortable capacity margins for live load. Modern techniques have highly reduced the costs of construction, but they are developing faster and faster and a lot of bridges are built before their behaviour can be tested by time. Moreover, the increase of heavy traffic, the attacks from the environment, the use of salt spreading in winter, the growing strains allowed by codes are factors affecting durability.

Nevertheless most of the initial defects of a new technique are later overcome. Moreover, modern structures do not necessarily require complete reconstruction, because the foundations, piers and abutments can usually be retained. Thus, while some existing bridges may need replacement after 40-60 years, a design life of the order of 100 years is generally considered to be attainable by modern structures.

2. THE CHALLENGE OF THE DESIGN

New techniques and innovation tend surely to lower construction costs in high proportions, and make it possible to have more ambitious construction programmes, with a fixed possible expenditure. But the audacity of the innovator must be accompanied by an equal prudence. As an example the five bridges built on the Marne River by Freyssinet in 1949, which were among the very first prestressed bridges, are nowadays in good condition, while some other prestressed bridges built later required rather substantial repairs, as we just have seen.

The experience from many repairs shows that some additional but limited expense during the building period would have later saved heavy expenses. Moreover, repairs are not always able to restore to the structure its normal life expectancy. One might say, as a figure of speech, that one dollar wrongly saved in the design generates ten dollars needlessly spent on site, and that ten dollars wrongly saved during construction generates hundred dollars of extra repairs during the lifetime of the structure.

So the challenge to the designer consists in avoiding false economies, while saving all what is possible through technical progress and skilled design. Intelligent design indeed is quite different from blind application of technical prescriptions or rules. It is fairly difficult because the designer has to think of many things : general design, detailing, possibilities of future disorders, accessibility for inspection and repair, etc. In the case of a somewhat innovative structure, he must imagine how this one will be working and find appropriate calculation models.

Usually the owner of a future structure chooses a particular designer, consultant or official, according to proficiency criteria, which he cannot always appreciate with a certain accuracy, or he even selects him according to quite other criteria, for example the amount of the required fee or the geographical proximity. In our opinion, this way of acting ought to change somewhat. The importance of good design for the bridge durability does not suffer to emphasize other criteria without sufficient regard to professional skill.

For some years now in France, and for fairly large structures, of course, we usually choose two or even more designers who will work together, splitting the whole task between themselves under the direction of one of them. This makes it possible to benefit from the specific experience and proficiency of several designers. It also allows one to make two and even several complete designs for a given bridge. So it is possible to test by the competition between contractors which is the better structure from the economical point of view, and to promote progress with all the required care. Design fees indeed are very small, compared with the benefits to expect from better designs.

Another solution equally used is for the owner choosing another consultant than the one in charge of the design in order to advise him, i.e. in fact to propose to the designer improvements to the design. We are convinced that the greater technical difficulty of large modern bridges is a valid reason to give greater care to obtaining a better design, as much in its main features as in all its details. Durability is at this price.

3. THE CONTRACTS

For contracts the same balance as for design is to be kept between security, which is a condition of durability, and risk, which is the counterpart of technical progress. A source of progress consists in allowing the contractor to present some alternate features to the design, in order to adapt it to his own equipment and building methods, or even in some cases to propose an alternative. The alternate features permitted by the competition rule may be more or less extended, but in our opinion it is desirable to leave a margin to the contractor, considering the importance of building methods in modern bridge design.

The counterpart of this intervention of the contractor must *not* be a reason to reduce the requirement of proficiency and skill concerning the main designer. On the contrary, it is necessary to have the detailed design proposed by the contractor entirely verified and recalculated by the consultant. Quality and durability require this care in the perfecting of the design.

We have concentrated on the quality of the design, but a similar effort is to be made to use the quality assurance methods in the whole building process. The professions of construction are not yet aware enough of the necessity of improving their working methods, as it is now being done in industry. This ought to be even more obvious for bridges, the life expectancy of which is much longer than for industrial products.

4. SMALL BRIDGES

Each small structure cannot be designed with the same luxury of care as with large structures. Nevertheless small bridges are much more numerous than the large ones, and the asset they constitute is higher. Therefore, the problem of their durability is as important as for the latter. Moreover, the contractors who build small structures generally are less skilled than those who build the large bridges.

So the major point concerning small bridges is simplicity : simplicity of design, simplicity of building. This is the best guarantee of their durability. A good solution to obtain this simplicity consists in standardizing them. Prefabrication may accompany this standardization. This last solution is used for example in Belgium, and in the cold countries, for obvious reasons of climate.

In France both the economical conditions and the climate make it possible to build small structures with cast in-situ concrete. So the major part of the standardized small bridges is composed of slab decks. Their design is quite simple and fast, due to the fact that the calculation of the standard slabs is made at low price in a special public office using specific computer programs.

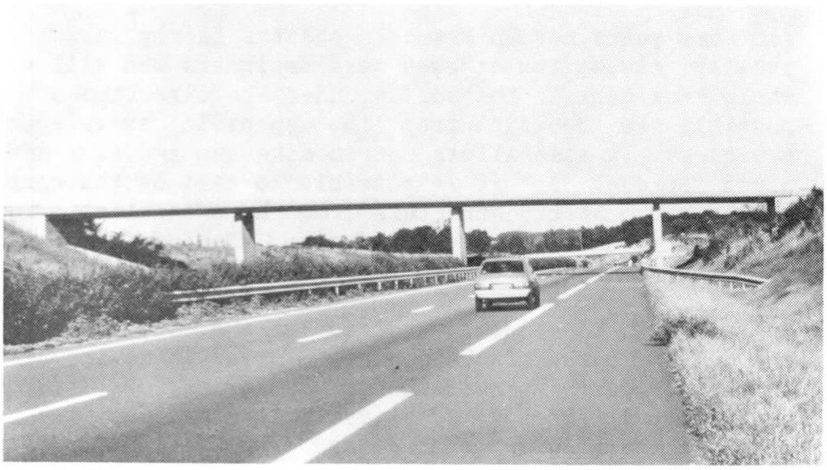

Fig. 8 : Simplicity of the slab deck for common bridges

These slabs are very strong structures, since in more than twenty years thousands of this type of bridge have been built, and their durability appears to be excellent.

5. BRIDGE MANAGEMENT

The different countries are now more aware of the necessity of promoting bridge inspection and maintenance policies, in order to obtain sufficient durability of this considerable asset. Prevention is more efficient and less expensive than subsequent repairs. Bridge inspection rules and bridge inspection manuals have been developed in several countries. Another step will consist in developing a bridge management system in order to obtain the best efficiency of public repair expenditures.

But an important aspect of maintenance concerns the inspection equipment. Large progress has been done in this field, but much more still remains to be done. For example, testing the load carrying capacity of reinforced concrete bridges by calculation would require further progress, in order to be able to detect the diameter and the location of all reinforcing bars. Devices exist nowadays, which can detect some details of the reinforcement, but not all the desirable ones.

6. CONCLUSION

It is now the moment to conclude this brief survey of some general aspects concerning bridge durability. The latter is an essential requirement for bridges perhaps even more for them than for other products of human activity, because the considerable asset they represent cannot be replaced very quickly, due to limitations of public budgets.

Now it seems that the main efforts of technical progress up to the present have tended to reduce the cost of construction. Without renouncing this purpose, it will be necessary in the future to bring an equal care to quality, which is an essential factor of durability. Taking into account the greater complexity and boldness of modern bridges, this may imply some change in enginering practice concerning design and contracting procedures. Finally, the better care now brought to maintenance certainly must be more developed.

Examining all these points is the object of the present Symposium. I hope it will come up to the participants' expectations.

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PRESENTATIONS

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Durability Research as an Investment Strategy

Recherches sur la durabilité pour une stratégie d'investissement

Dauerhaftigkeitsforschung als eine Investitionsstrategie

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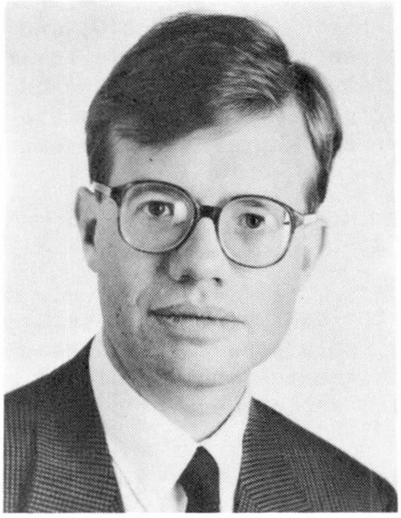

Jan Bröchner, born 1948, received his doctorate from the Royal Institute of Technology in 1978. His present research interests concern the economics of construction and management of existing structures.

SUMMARY

If the optimality of a structure as well as of its programme of maintenance and renewal is measured by total present value, any addition to our knowledge of the durability of materials and components should be measured by its contribution to this value. It follows that the economic consequences of research projects on durability can be studied with a strategic perspective.

RÉSUMÉ

Si l'optimum économique de la conception d'un ouvrage ainsi que de son programme d'entretien et de renouvellement est mesuré à sa valeur totale actualisée, tout progrès de nos connaissances concernant la durabilité des matériaux et composants doit être évalué en fonction de sa contribution à cette valeur. Par conséquent, les effets économiques de projets de recherche sur la durabilité sont étudiés dans cette perspective de stratégie.

ZUSAMMENFASSUNG

Wenn die Qualität eines Bauwerks sowohl am Bauwerk selbst als auch am zugehörigen Programm für Instandhaltung und Erneuerung als Gegenwartswert beurteilt wird, so ist jede Vermehrung unserer Kenntnisse bezüglich der Dauerhaftigkeit von Baustoffen und Komponenten nach ihrem Beitrag zu diesem Wert zu beurteilen. Auf dieser Grundlage werden die wirtschaftlichen Folgen von Forschungsprojekten über Dauerhaftigkeit mit einer strategischen Perspektive untersucht.

1. INTRODUCTION

1.1 Profit maximization

The choice of materials and components as well as maintenance and replacement policies over time can be formulated as a profit-maximization problem for the owner of a structure or group of similar structures. Let $Q(t)$ be the quality of the structure at time t and assume that the stream of revenues R from the use of the structure is a function of quality, $R = R[Q(t)]$. The stream of costs $C(t)$ is initiated by construction costs $C(0)$ and after $t = 0$ caused by maintenance and replacement expenditure. The time horizon for the maximization is $t = T$, which may or may not be the date of total demolition of the structure. Finally, the rate of discount which expresses the time preferences of the owner is taken to be ϱ, and we can express the present discounted value as follows:

$$\int_0^T \{R[Q(t)] - C(t)\}e^{-\varrho t}dt$$

Over the chosen time period, the maximum present value is to be achieved. By application of control theory, solutions can be found, giving not only the optimal maintenance paths but also the optimal timing of discontinuous increases in $Q(t)$, i.e. partial replacement [1].

1.2 Revenues

For most structures, the stream of revenues is not immediately known. Even when the structure or the services of the structure appear on the rental market or is subject to user fees, the relation between R and $Q(t)$ may be difficult to ascertain. This is especially true of the influence on revenues of small changes in functional quality, although the development of a range of quantitative indicators for the functional condition of road surfaces shows what is feasible for large systems of infrastructure under a unified management [2].

However, many of the structures lacking an easily identifiable stream of revenues are managed on the principle of approximately constant quality over time. The optimization is then equivalent to a minimum life-cycle cost approach [3].

1.3 Costs

Costs of maintenance and replacement may appear easier to predict than the effect of quality changes on revenues. An exception is when there is a possibility of cumulative damages occurring on failure of a structural part, as with reinforced concrete [4]. Risk analysis based on a mapping of failure mechanisms, their wider consequences as well as maintenance and replacement options, should then be performed, provided that the analysis in itself gives a positive net contribution to the expected present value.

1.4 Data and level of application

In current practice, direct optimization is usually ruled out because of lack of data, discontinuities and complex patterns of interaction. Depending on the type of structure, obstacles to data access arise from fragmented and competitive ownership or high costs of collecting data for a unique structure subject to a unique environment. To some extent, advances in information technology will improve the availability of data. Regarding discontinuities and complexities, the theory as it is can only be applied at a suitable level of aggregation: applied to a system of several structures rather than the individual one, so that behaviour over time and resource flows into maintenance can be approximated by continuous functions.

2. THE CONCEPT OF DURABILITY

2.1 Durability and service life

The related concepts of durability and service life are properly of interest only for the special case of materials and components that either perform their function or fail to do so. However, as concepts they provide a convenient focus for studies into the long-term behaviour of materials and components, not least for the large group of structures where there is a technology for recurrent maintenance as well as partial replacement, prolonging their lives and postponing total replacement. In practice, we encounter a long scale from partial replacement to total renewal.

Viewed under the appropriate magnification, any maintenance activity including cleaning or repainting falls into one or several of the categories of removal, replacement or addition of materials (or components). By disaggregating, a level can always be found where service life is a meaningful concept. Unfortunately, there are few cases where this level allows direct optimization. Nevertheless, research into durability of materials and structures is important because it focuses on $Q(t)$, or in other words, performance over time, in the form of degradation studies.

2.2 Degradation

Degradation of materials and components under the influence of various degradation factors [5] is the main object of durability studies. A variety of methods is available to the researcher, field studies, accelerated aging and the parallel development of theory, as in the case of concrete structures [6]. Degradation in an existing structure gives rise to maintenance and replacement needs.

3. MAINTENANCE AND INSPECTION

3.1 The quality path over time

The choice of maintenance policies is simplified in the constant quality case, although the determination of an optimal period of renewal will depend on the maintenance cost function. We should avoid looking at maintenance as an operation which in principle just restores the $Q(0)$ of the structure; in many cases, there is a technologically inseparable combination of activities intended to raise the load-bearing capacity or other qualities above the initial level. Conversely, it may be profitable to reduce the original quality level and remain at a lower level. Thus, from the economic point of view, the $Q(t) = Q(0)$ case is fortuitous or, if imposed as a restriction on the management of the structure, sometimes more due to difficulties in information handling than to functional analysis of the use of the structure.

Formulating the optimal maintenance path problem with the help of the theory of optimal control usually implies that the present value maximization is supplemented with a quality change restriction, where maintenance enters as a control variable:

$$\dot{Q}(t) = m(t) - \delta(t)Q(t)$$

where $m(t)$ is a measure of maintenance resources consumed at time t and $\delta(t)$ is the degradation or depreciation function. If $m(t) = 0$ and $\delta(t) =$ constant, the quality of the structure decays exponentially, as in Fig. 1. The two degradation paths based on δ_1 and δ_2 show the possible effect of durability research: either an improvement in the material so as to attain the δ_1 level or a narrowing of the uncertain area between δ_1 and δ_2, reducing the risk in choices during the design stage. In Fig. 2, the effect of durability studies on materials or components with a pattern of sudden failure is shown: either a prolongation of service life from t_2 to t_1 or a more narrow interval of life being predicted.

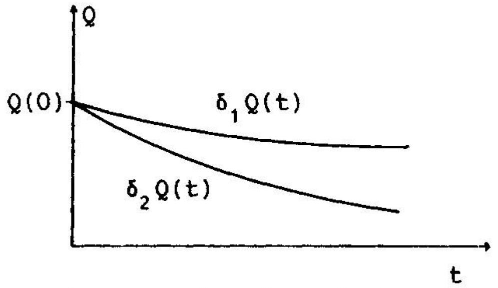

Fig. 1 Successive degradation

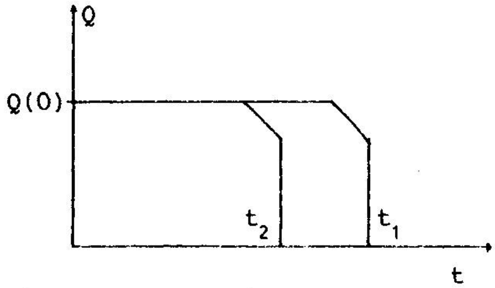

Fig. 2 Sudden failure

3.2 Preventing damages

Preventive maintenance may raise the present value if there is significant irreversibility in processes which start when quality declines below a certain level. Structural collapse is a typical example, roof leakage a less dramatic one. Optimal strategies for preventive maintenance can be condition-based, including routine inspection, or time-based (periodical), which should be the choice when the cost of inspection exceeds the expected increase in total present value associated with the structure.

The development of durability studies increases our ability to recognize and anticipate irreversible quality changes in a structure. Such knowledge can be translated into alternative patterns of inspection or even as the continuous monitoring of decay through the information network of an 'intelligent' building. Inspection, the choice of intervals, the resources devoted to it and consequential savings, should be seen as a part of the maintenance strategy and as a part of a joint optimization problem for the assignment of scarce resources [7].

4. EVALUATING DURABILITY RESEARCH

4.1 The economic value of research findings

The findings of durability research can be used in three contexts:

(a) in the materials industry, when developing new materials or combinations of materials,

(b) at the design stage, leading to better choices of materials and components;

(c) after completion of the structure, when choosing or modifying strategies of inspection, maintenance and replacement (partial or total).

For a given type of research projects, the relevant set of structures with reasonably similar technology and similar environmental conditions must be identified first, both existing structures and projects in the foreseeable future. Secondly, the situations of economically important choices in the three contexts (a - c) should be analysed. The analysis comprises an assessment of the range of substitute materials, both at present and over the lifetime of the structure, alternative methods of maintenance and the risks involved in deferred maintenance and replacement, at the present state of knowledge and as expected from the project in question. After that, present value analysis should be considered. As an example, the design choice between two paints A and B with different degradation properties, following as most surface materials do the pattern of Fig. 1, can be based on the ensuing maintenance costs [8]. The calculation is more complicated when assessing the value of risk reduction (in terms of Fig. 1, that the true degradation pattern is δ_1 and not δ_2, e.g.), and in the situation of Fig. 2 with sudden failure, the emphasis of the analysis is shifted towards a probabilistic approach, identifying risks and their relations. Finally, the potential for spin-off effects owing to new, more fundamental insights into decay mechanisms with a wider application should be assessed, although this is mostly a matter of intuitive judgment.

4.2 Recipients of benefits

Like most forms of research with a fragmented pattern of beneficiaries, it is difficult to finance activities by user fees according to benefits. Where there are single users of the information, such as materials producers with local monopolies in a uniform environment, or the structures - like many bridges - form part of a technical or legal monopoly, something approaching an efficient allocation of research resources should appear spontaneously. However, if materials and structures are sold in markets, where durability information is costly to transfer from seller to buyer, there will be insufficient incentives to improve the long-term performance through research. A textbook solution, when inspection and quality control costs are high in market transactions and there is a considerable risk of latent defects, is vertical integration through joint ownership of the stages of production. Since this is seldom the case for buildings and the physical infrastructure, cooperation and government support is needed to reach an efficient level of durability research.

The case of the monopoly producer of a material holds another complication. If the extended life of products reduces the total materials consumption per annum, and there are economies of scale in production, some of the returns to scale will be lost when sales decline [9]. On the other hand, the superior durability may lead to a wider market for the product, which compensates for the immediate loss of volume.

4.3 Quality of research

Especially where component failure may lead to severe damages, questionable validity and reliability of research findings introduces an important stochastic component. Also, it is often so that durability findings are loosely applied to a slightly different material subject to slightly different environmental action. Care should be taken that research methods and presentation of findings minimize additional risks in application. Expert evaluations of research projects is one of several ways to monitor the level of quality in durability studies [10].

5. AN INVESTMENT STRATEGY

Returning to the initial present value maximization, we can look at durability studies in the same perspective. The body of knowledge in the field of durability is treated analoguously with the structure, and the successive maintenance inputs have their parallel in additional research findings. Due to new discoveries and the general development of technology, parts of the existing knowledge are rendered useless, similar to the successive deterioration of Fig. 1. The revenues from durability research have to be derived from the savings made in materials production and in the management of erected structures.

In practice, the total optimization implied by this approach has to be replaced with a strategy of suboptimization: a set of priorities for investment in durability research should be established working backwards, primarily identifying alternative methods of maintenance and replacement, analysing how great the economic losses can be when durability characteristics of materials and components are known imperfectly.

6. CONCLUSION

Within the framework of present value analysis, it is necessary to consider both the long-term revenues and the costs associated with structures. A flexible view of maintenance and programs of partial replacement can then be applied and the value of durability research assessed in this context. For the specific case of sudden failures with major irreversible consequences, risk analysis should be used according to priorities based on an identified potential to contribute to overall present value. The lack of incentives for research is traced partly to the costs of knowledge transfer between sellers and buyers when markets for materials and structures exist. A tendency to underinvest in durability studies should be met by research cooperation and strong government funding.

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Role of Codes in Bridge Durability

Rôle des codes pour assurer la durabilité des ponts

Die Einführung von Normen zur Verbesserung der Dauerhaftigkeit von Brücken

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SUMMARY

If durable bridges are to be produced, reliability and serviceability must be addressed in new design codes. Probabilistic methods enable this to be done by calibrating serviceability conditions to agreed levels. Rehabilitation codes are lacking in reliable data for satisfactory calibration. Tendering methods affect durability levels. Four alternative tendering methods are reviewed. The build/operate/transfer method appears likely to produce the most durable structures.

RÉSUMÉ

Afin d'augmenter la durabilité des ponts, il faut envisager de nouveaux codes de projet. Ce but peut être atteint par des méthodes de probabilité, en calibrant les conditions de service à un niveau acceptable. Les codes de réfection manquent de données valables pour leur calibrage satisfaisant. Le système des soumissions a un effet sur les niveaux de durabilité des ponts. On considère quatre types de soumissions; celle qui semble devoir donner les meilleurs résultats quant à la durabilité des structures, combine la construction, l'opération et le transfert final au propriétaire.

ZUSAMMENFASSUNG

Um bei dem Bau von Brücken einen hohen Grad an Dauerhaftigkeit zu erreichen, müssen in bezug auf Zuverlässigkeit und Instandhaltungsmethoden neue Normen geschaffen werden. Bestimmte Prüfmethoden erlauben es, gewisse Normen für die Brückeninstandhaltung nach bestimmten Richtlinien festzulegen. Reparaturvorschriften geben keine genauen Auskünfte über zuverlässige Instandhaltungsmethoden. Die üblichen Ausschreibungsmethoden beeinträchtigen die Dauerhaftigkeit. Vier verschiedene Methoden werden besprochen. Die Bauen/Betrieb/Übertragungsmethode scheint den höchsten Grad an Dauerhaftigkeit und Zuverlässigkeit beim Brückenbau zu erreichen.

1. INTRODUCTION

Durability is described as the ability of the structure to maintain its level of reliability and serviceability during its lifetime. In the past, durability has been considered mostly in terms of serviceability items, such as cracking and spalling of concrete, corrosion of steel, and limiting maintenance costs. These items were considered as primarily the preserve of construction specifications, site inspection and quality control, rather than that of design codes. Such codes might specify minimum concrete cover, for instance, and other items of good construction practice, but could not adequately address the question of reliability as these older working stress codes were deterministic rather than probability based.

With the introduction of probabilistic limit states bridge codes around 1980, lifetime reliability has become one of the most significant areas for code development. If the required statistical data is available, load and resistance factors can be calibrated to achieve target safety indices at various serviceability limit states as well as the ultimate limit state. Durability has thus become very much an area of interest for design codes.

The 20 year boom in new highway and bridge construction peaked in North America and a number of European countries in the late 1960's. As many of these bridges are now ageing and require extensive maintenance, there has been an increased interest in bridge rehabilitation in the last ten years. This interest initially focussed on repair techniques and materials rather than rehabilitation design, as codes did not address this issue. If rehabilitation is to be cost effective, much work needs to be done on putting rehabilitation design in the same reliability based context as new designs. Much more data is needed on the life expectancy of rehabilitated bridges before this can be completed, but the third edition of the Ontario Highway Bridge Design Code (OHBDC) will have a new section on rehabilitation design when issued late in 1989.

Bridge codes have generally been written to cover frequently built bridge types in the short and medium span range. For long span bridges special design criteria are usually prepared which may not address durability adequately, particularly if probabilistic data is not available which may be the case for a unique design. It is in the long span bridge range that different tendering methods are likely to be used, such as alternative designs, design/build proposals, and more recently build/operate/transfer (BOT). These various methods can produce in themselves wide variations in durability. The latter method, BOT, holds promise for a high level of reliability and serviceability, however, and a current Canadian example is presented.

2. DURABILITY ASPECTS IN DESIGN CODES

Whatever basic code philosophy is used, all bridge design codes should precribe details to ensure ease of maintenance, and specify design details that are considered good practice and likely to produce durable structures. The OHBDC 1983 explicitly addresses maintenance and durability aspects, and some of its provisions will be identified as typifying what can be covered for bridges in a corrosive environment, regularly subject to winter salting.

The components most susceptible to deterioration have been concrete bridge decks, expansion joints and bearings[1]. The minimum slab depth is specified as 225 mm with a minimum cover to the top reinforcing steel of 50 mm. Placing tolerances for reinforcement are given which have to be allowed for in setting the dimensions on the drawings to ensure that the minimum cover is achieved in the field. The OHBDC commentary references the use of epoxy coated reinforcement and membrane waterproofing for decks. Deck drainage and drip detail requirements are given, with downspouts to protrude below soffit level to keep salt water off the superstructure.

Sealed deck expansion joints are normally used, but as they frequently leak, the use of continuous spans to minimize joints and thus improve durability is encouraged. Seals have to be replaceable, and must be set below the riding surface to reduce wear. If the joints eventually leak, access is needed between the abutment ballast wall and the deck for cleaning, and a minimum gap of 200 mm is called for to enable this to be done. Bridge seats must have a grade of at least 5% so water will drain away from bearings. Bearings have to be accessible for inspection and maintenance and be replaceable without damage to the structure and without removing anchorages permanently attached to the structure.

To facilitate inspection and maintenance of steel or concrete box girders, and enable interior formwork to be removed, access openings have to be provided for each cell, and have closely fitting hinged covers. Such girders shall not contain sewers or water pipes inside them due to the possibility of breakage or leaking, and subsequent girder deterioration or danger of collapse. Gas and oil pipelines are prohibited from all highway bridges on account of fire and explosion hazards.

All these items may appear rudimentary, yet they have to be specified at the design stage, and need to be included in the design code to ensure implementation, otherwise the durability of the built structure may be compromised.

3. RELIABILITY BASED CODES

The move in recent times towards reliability based codes, enables codes to be calibrated to produce relatively consistent safety levels for bridges. This calibration work has generally concentrated on the ultimate limit states [2], but can equally well be applied to serviceability limit states, thereby providing another means of establishing durability levels for structures at the design stage. For the 2nd Edition of the OHBDC in 1983, such calibration was carried out for the serviceability limit states of cracking, vibration, fatigue and permanent deformation [3], as well as ultimate limit states.

The design equation for each specified limit state is:

$$\phi R \geqslant \text{total factored load effect}$$

where ϕ is a resistance factor, R is the nominal resistance, and total factored load effect is the sum of the product of the nominal loads considered multiplied by their corresponding load factor. The calibration process used was the calculation of load and resistance factors, using second moment level-2 reliability analysis [4], to obtain a reliability index close to the preselected target value. The reliability index β is a measure of safety, such that:

$$\beta = \frac{\bar{R} - \bar{Q}}{\sqrt{\sigma_R^2 + \sigma_Q^2}}$$

where $\bar{R}$ and σ_R = mean resistance and its standard deviation and $\bar{Q}$ and σ_Q = mean load effect and its standard deviation.

The target β selected for ultimate limit states was 3.5. The serviceability limit states can be reached more frequently and lower β values can thus be selected. For example, for cracking of prestressed bridges, a target value of $\beta = 1.0$ was used, which relates to concrete cracking under live load once a week. This frequency of crack opening was considered acceptable, considering the possible fatigue of the prestressing strands and the possible corrosion of strands due to the entry of aggresive salt water. From the durability point of view it should be noted that Ontario practice calls for a full waterproof membrane and asphalt wearing surface over prestressed decks in addition to the serviceability controls on the concrete.

The target reliability values can be selected according to the type of structure and its importance. For instance, for elevated transit structures, where service must be maintained at all times, and no alternative routes are available, higher target β values of 4.0 and 2.5 for ultimate and cracking limit states have been proposed [4].

4. LIFETIME SERVICEABILITY

Based on the history of bridge replacements in North America, a typical design life expectancy would be 50 years. Bridges have become deficient due to functional or geometric inadequacies, serious structural deterioration, or insufficient load carrying capacity due to an increase in vehicle weights. Most bridges over 30 years old have required significant rehabilitation work. With calibrated limit states design codes and a better understanding of the design, construction and operational needs for more durable bridges it is expected that new bridges will show better performance. It is unlikely, however, that any bridge will achieve its 50 year lifetime without some rehabilitation work being needed. If a bridge is to maintain the designed level of reliability and serviceability during its lifetime it is desirable that the code to which it is designed also includes provisions for load capacity evaluation and rehabilitation.

Evaluation and rehabilitation aspects have traditionally not been part of design codes. The first two editions of the OHBDC have covered bridge evaluation, and the third edition will have a new section on rehabilitation. The limit states format is ideally suited to evaluation and rehabilitation, as the actual bridge can be surveyed and the design values of load and resistance factors modified as appropriate. These factors can also be adjusted to suit the anticipated future life of the structure, which is unlikely to be as long as the 50 year life on which the design values were based. By using these methods the load carrying capacity will usually calculate higher than that obtained by applying new design provisions to the evluation process.

When rehabilitation design is required, the new bridge design provisions are not usually suitable, and rehabilitation code sections should again reflect changed loading conditions, structure condition and anticipated future life. The OHBDC will have three rehabilitation categories according to an anticipated future life of greater than 25 years, 10 to 25 years, and up to 10 years. Each category will have its own prescribed load factors. It will be difficult to do a comprehensive calibration of the rehabilitation load and resistance factors at this time, as life expectancy of various rehabilitation techniques is hard to establish. When enough rehabilitation data has been collected, rehabilitation design can be put on the same probabilistic basis as new structures. A code can then consistently address new design, evaluation and rehabilitation to increase the probability of maintaining a uniform level of serviceability throughout the life of a bridge.

5. TENDERING METHODS AND DURABILITY LEVELS

5.1 Background

Most methods of tendering for bridge construction give little incentive to produce the high quality work that will enhance lifetime durability. They usually award to the bidder with the lowest construction cost.

In North America this price is prepared using full design drawings and specifications prepared by the owner or his consultants. Standard design codes are usually applied. On some major projects alternative tendering methods have been used, usually on long span bridges which are beyond the range for which standard design codes apply. Special design provisions have to be prepared, including those addressing durability. The different methods of tendering can have a major impact on the likelihood of obtaining durable bridges, and four methods will be compared from this perspective.

5.2 Single Design Provided by Owner

With this method all contractors bid on the same design, with full drawings and specifications issued, and no provision for changes, except perhaps by applying value engineering after the contract is awarded. The design requirements and construction specifications need to be comprehensive and must be supported by a major quality assurance program by the owner, as the contractor, in order to obtain the job, has to bid providing no more than the minimum specified quality.

The method works reasonably well for large public authorities when building short and medium span bridges on a regular basis. Most highway departments in North America use this method for their bridges, unless the contract value is high enough to warrant going to alternative designs.

5.3 Alternative Designs Provided by Owner

On a large bridge project when it is difficult to select the single most economical design, the owner may provide two or more designs for competitive bidding. The controls needed for durability noted for the single design method are equally necessary under the alternative design method, as the award is still based on the lowest bid, with full documents provided by the owner. There are two further considerations, however, for the alternative design approach. As this method is usually applied to long span bridges, beyond the typical maximum span of about 150 m for which standard design codes apply, special design criteria may need to be prepared covering durability aspects of long spans and possibly unusual types of bridges. The other consideration comes at the design stage, and relates to preparing designs of equal durability, and as far as possible equal maintenance costs over the lifetime of the bridge. As the award is made on lowest construction cost, rather than lowest life cycle cost, the process makes sense only if each alternative is equally acceptable to the owner, which implies equal reliability and serviceability.

5.4 Design/Build with Designs Provided by Contractor

This is the method commonly used in Europe, but has been used only recently in North America. The contractor prepares his own design and bids on this, using overall requirements supplied by the owner. The method is usually applied to large projects or long span bridges, and particular attention again must be paid to design criteria preparation. This may be even more important than under the alternative design approach, as the owner does not know what types of bridge will be designed by the bidders. The specially prepared design criteria thus have to cover a very broad range of structure types and materials to ensure comparable durability. As the award is again based on lowest construction cost, the concerns about equal serviceability and maintenance costs given under the previous method are equally applicable under the design/build approach, but are probably harder to achieve.

The design/build method was used for the retractable roof stadium "Skydome" in Toronto due to open this year. A two phased procedure was followed with approval of technical concept as a first stage producing four finalists. The competing roof types being now known, the final design criteria were prepared before moving up to the second stage of pricing. This stadium roof, with barrel arches spanning up to 200 m is fully exposed to the elements when retracted, and is in many ways more similar to a steel bridge than a building. One interesting aspect of the design criteria, addressing reliability and durability, was the requirement that the roof structure had to stand up after all members within a vertical cylinder of diameter 4.5 m located anywhere in the roof were removed. This requirement produced a highly redundant winning design with a multiplicity of load paths, probably the best way of ensuring a high level of reliability.

5.5. Build/Operate/Transfer (BOT)

The Northumberland Strait Project, a 13 km long bridge linking the Provinces of New Brunswick and Prince Edward Island is the first application of the BOT method in Canada. Developers are required to finance, design and build the toll facility, operate it for 35 years, then transfer it to the Federal Government. The competition has three phases, initial qualification of developers, acceptance of concept proposals, and finally the price proposals. The project is presently held up for a year, following the completion of the second phase, pending a further environmental study.

Design criteria were written by the Federal Government before the second phase, but left many tasks to the developer, such as calibrating the criteria for the ultimate limit states to a target β value of 4.0, carrying out wind tunnel tests, and performing ship collision risk analysis. To the extent possible durability requirements were given for steel and concrete bridges, with the objective of using the best possible techniques. The eventual owner prescribed a 100 year life, in the aggressive environment of an ocean crossing with high winds, wave action, and large ice forces. To increase the chances of achieving this lifetime, and of having a bridge in good condition at the transfer stage, the use of salt as a deicer on the roadway is prohibited. The developer, as the operator, has to use other methods such as urea or CMA.

An important aspect of the BOT method is that the developer is just as interested in durability as the owner. In fact the developer may aim for durability in excess of the prescribed minimum in order to avoid a major rehabilitation cost before the transfer date. The proposals are in effect based on a total 35 year lifetime cost to the developer, as the project will not be awarded on construction cost, but on the basis of the lowest government annual subsidy requirement.

6. CONCLUDING REMARKS

Techniques are now being developed so that design codes for new bridges can properly address lifetime durability. More information on the expected life of repaired components is required before similar progress can be made on rehabilitation design codes.

The level of durability may vary with the contract tendering method adopted. The BOT method on major bridge projects holds the most promise for improved durability as it is the only method that makes long term durability a common goal for both owner and constructor.

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Durability, Defects and Repair of North Sea Structures

Durabilité, détérioration et réparation de plateformes en Mer du Nord

Dauerhaftigkeit, Schäden und Reparaturen von Tragwerken in der Nordsee

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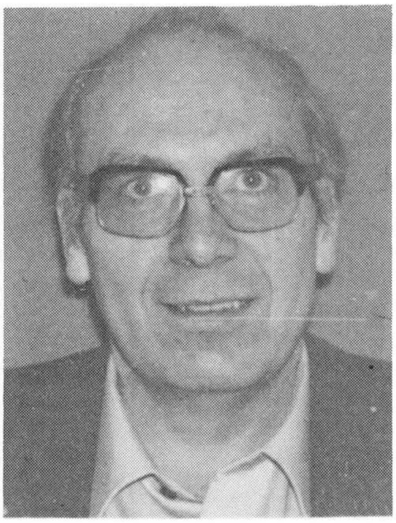

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SUMMARY

The paper gives a brief summary of the findings from the in-service inspections on North Sea offshore structures. Needs and methods for maintenance and repair are outlined. The causes of the observed deficiencies are discussed. Main issues for the steel structures are accidental impact and other unforeseen overloading. Other significant findings concern cracks and the cathodic protection systems. The concrete structures have performed well. A few localized structural deficiencies required attention. On some older platforms the mechanical systems gave some concern.

RÉSUMÉ

L'article résume les observations faites lors d'inspections de plateformes en service en Mer du Nord. Les exigences et les méthodes d'entretien et de réparation sont résumées et les causes des problèmes observés sont discutées. Les éléments saillants concernant les structures métalliques sont les chocs accidentels et autres conditions de surcharges non prévues. D'autres remarques d'importance s'adressent à la fissuration et aux systèmes de protection cathodique. Les structures en béton ont un bon comportement, sauf pour quelques défauts localisés qui exigent un soin particulier. Certaines plateformes plus anciennes ont connu des problèmes de systèmes mécaniques.

ZUSAMMENFASSUNG

Der Aufsatz gibt eine kurze Zusammenfassung der Ergebnisse der Betriebs-Inspektionen von Offshore-Tragwerken in der Nordsee. Bedarf und Methoden für Wartung und Reparatur werden beschrieben. Die Gründe für die gefundenen Mängel werden diskutiert. Die wichtigsten Probleme für Stahltragwerke sind Stossbelastungen und andere unvorhergesehene Überlastungen. Andere wichtige Erkenntnisse betreffen Risse und das kathodische Korrosionsschutzsystem. Die Betontragwerke erfüllen ihre Funktion gut. Wenige lokale Mängel der tragenden Bauteile erfordern Aufmerksamkeit. Bei einigen älteren Anlagen bestehen Probleme bei mechanischen Komponenten.

1. OFFSHORE STRUCTURES IN THE NORTH SEA

1.1 Number and types of structures

Since the first steel structures were installed in the shallow Southern North Sea in the sixties more than 100 steel jackets and altogether 20 concrete structures have been installed. The water depth record is held by the 216 m deep Gullfaks C platform installed the summer 1989.

Two main types of fixed structures are considered.

1) Steel jackets i.e. space frames built up of slender tubular members with piled foundations.

2) Concrete gravity base structures composed of a voluminous caisson base with 2-4 large diameter towers projecting up throught the water surface and supporting the deck. Other platforms consist of one large central shaft surrounded by a perforated wall.

This paper covers all 20 concrete structures, and about 30 steel platforms including all on the Norwegian Shelf. Smaller structures such as loading buoys and floating units are not covered.

1.2 Environmental exposure

The environmental exposure in the North Sea climate is characterized by large static and dynamic wind and wave loads, fatigue exposure and corrosivity. Typical wave heights with one month recurrance periode is in order of 20 m whereas the design wave (100 years recurrance periode) is about 30 m.

1.3 Materials and workmanship

Low carbon structural steels with yeld stress typically 360 MPa are specified for the steel structures. All essential welds are 100% X-ray tested. The concrete structures have concrete qualities in the range from C45 to C60.

1.4 In service inspection

The oil companies as well as the Governmental Authorities require all significant points of the platforms to be inspected on a yearly or four-yearly basis. In addition special surveys are made whenever found appropriate e.g. after possible damage. In service inspection of both steel and concrete structures are based on visual inspection. Steel structures are also subject to NDT inspection i.e. MPI, ultrasonics and potential measurement of the corrosion protection. The reliability and detectability of small deficiencies by the NDT methods under water has been questioned.

2. STEEL STRUCTURES

2.1 Cracks in welds

Cracks have been observed on a majority of the platforms investigated. The maximum length was 750 mm. In platforms where the cracks cannot be related to improper design or overloading, cracks were observed in 6-7% of the welds investigated. About half of these cracks were removed by grinding to a depth of 2-3 mm. Occationally much deeper grinding has been carried out.

Particular investigations have been made in dented areas and at burn marks. The crack frequency was much higher in these areas, but a majority of them was removed by grinding.

Cracks not removed by grinding have been kept under observation to see possible propagation. Cracks caused by loads inadequately designed for only, were found to propagate as expected for a fatigue crack. There are, however, cases where cracks turned up again after apparently successfull grinding removal.

Several crack cases required comprehensive repair work i.e. by welding, bracing removal, construction strengthening etc.

2.2 Dents and deformations

Dents were observed on the majority of steel platforms. In most cases they are accompanied with deflections of the member in question. Dent observations are evenly distributed over the heigth of the platforms.

As mentioned cracks are frequent in the dented zones. The cold working associated with the denting can result in surface brittleness. The irregular distorted geometry gives stress peaks which promote fatigue. There are cases where bracings have broken as the result of these mechanisms.

Where repairs were found necessary, the damaged members have been cut out and new sections have been introduced by welding and clamping. Dented struts have had their capacity in compression reestablished by filling the tubular member with grout.

Local strengthening has been accomplished by welding on doubler plates or by mechanical clamping fixed by bolts. Clamps grouted around the weakened structure is another popular repair method. In some cases stabilizing braces have been welded on to the deflected member to avoid further deflections. In other cases a paralell bracing have been welded to unload the damaged member.

The accomplishment of these operations under water, or even more complicated, in the splash zone is a difficult and expensive undertaking.

2.3 Corrosion

General corrosion is no major concern. Only a few platforms are found to be seriously attacked, and in limited areas only.

Pitting corrosion, on the other hand, is observed on the vast majority of platforms. Pitting 5 mm deep is not unusual. The extent and seriousness of the pitting vary significantly between platforms and between nodes. No particular elevation seems more exposed than others.

Most of the pitting seems initiated shortly after the platform installation, maybe before the necessary cathodic polarisation is established.

The cathodic protection systems seems rather vulnerable. Lost or loose anodes or other anode deficiencies have been observed on more than half the platforms.

Approximately half the platforms were also found to have too high electrochemical potential (i.e. more positive than -0.85 V to a silver electrode). On about one third of the platforms more anodes have had to be added after the installation. Metallic debris in electric contact with the platform increase the anode consumption. This kind of debris have had to be removed from all the platforms.

2.4 Sea floor erosion

Platforms installed on an erodable sea floor are normally designed assuming 10 ft seabed erosion. The platforms dealt with in this paper are mainly in water depths 70 m and more. Only one of these platforms experienced erosion exceeding the design assumption.

2.5 Marine growth

A majority of the platforms experienced a significant marine growth. The first platforms installed were not designed to resist the added hydrodynamic loads associated with the fouling and cleaning has been necessary. The design criteria introduced for later platforms seems to cover the growth observed in practice.

2.6 Conclusions, steel structures

A large portion of the significant structural findings concerned impacts from ships and dropped objects. Similar type of impact and overloading occurred during installation.

Another important reason for repair is static overload. In some cases the strengthening was necessary to cope with increased loads beyond the original design basis. In other cases the cause might be classified as design errors. Some more robustness to accidental impacts and other unforeseen loads might have been desirable.

Damages classified by "cracks" and "workmanship" cover a variety of different causes. A majority of these cases should probably have been avoided by better site supervision and control.

3. CONCRETE STRUCTURES

3.1 Leakages

Leakage through porous concrete, cold joints or encased pipes was only observed on some of the platforms constructed in the early seventies. Apart from the three cases mentioned below signs of leakage has only been observed by deposits and occasionally a humid surface. This always was a temporary problem and after a few months the seepage selfhealed. It is a general observation that cracks and porosities heal and seal by deposits ($CA\ CO_3$ and $Mg\ (OH)_2$) produced by reactions between the concrete and the sea water.

In one case the ingress of water was as large as $5m^3/h$. The ballast pumps could easily cope with this leakage rate. Without further precautions the leakage was observed to decrease with time and after three years the inner surface was completely dry.

Two minor leakages were sealed by injection with epoxy. A minor leakage through a burst plastic pipe was repaired by grout injection.

3.2 Foundation cracking

Cracks were observed in a cantilever wall supporting part of a foundation slab, presumebly caused by uneven soil reactions. The cracks were injected by epoxy and the surrounding sea floor covered by scour protection.

3.3 Precast formwork spalling

Some of the early platforms were fitted with a perforated breakwater wall. For convenient construction a precast concrete formwork was applied. On one platform this formwork spalled off in a limited area. The cover to the reinforcement was reduced more than formally acceptable. Repairs were done by replacing unsound concrete by cement mortar and covering the area with epoxy.

3.4 Impact damage

Five cases of significant ship impact have been reported.

None of them resulted in damage reducing the structural strength. Four impacts resulted in minor scratches in the concrete surface only, and no repairs were necessary.

One of the impacts caused through-cracking. A leakage of 5 litres/min was observed. This damage concerned one of the first platforms in the UK-waters. It was not designed to resist ship impact.

To avoid possible rebar corrosion the area was repaired by:

- epoxy injection of the cracks and cavities
- grouting removed unsound concrete
- epoxy coating internal and external surfaces.

Eight dropped object cases have justified subsequent structural investigations. Six investigations concluded no repair to be required. These concerned pipes of diameter up to 30".

A 36" spool piece of 10.5 t weight was dropped onto the caisson roof and punched through. The structure was found to be structurally sound and operation could continue without further measures. As the punched caisson was used as oil storage a repair was undertaken by grouting cement mortar into a prefabricated form filled with gravel.

A 36" drain caison of 20 t weight was dropped onto the caisson roof. The only consequence was some spalling of the concrete cover. Epoxy was injected under a formwork to restore the rebar cover formally required.

3.5 Seafloor erosion

One case of seafloor erosion has been observed. This concerned two voluminous concrete structures installed close to each other in relatively shallow water. The situation was stabilized by filling the eroded area with sand bags.

3.6 Conclusions, concrete structures

It is remarkable how well the concrete structures in themselves have performed in the hostile North Sea climate. No signs of material deterioration, rebar corrosion or other deficiencies have been observed.

A common feature of the findings on concrete structures is their local nature. The structural integrity of the platforms has never been jeopardized. Typical cases have been impacts from ships and dropped objects resulting in local cracking of the concrete.

Concrete platforms normally have comprensive mechanical systems for ballasting, water outlet, oil storage etc. On some of the early platforms these systems were not adequately designed. Material weakness and inadequate tightening of pipe penetrations have lead to a few leakages.

A general conclusion to be drawn is that the concrete structures required significantly lower expenditures for inspection, maintenance and repair than the steel platforms. The extent of the inspection tends to be reduced over the years as no findings are made under normal operating conditions.

TABLE 1

Reported repairs of North Sea Offshore Structures.

Repair, replacement and installation of new anodes, grinding of smaller cracks and marine fouling removal not included.

Causes	Steel structures	Concrete structures
Local foundation overload	-	1
Dropped objects	8	2
Ship impact	7	1
Workmanship	5	3
Abrasion by hanging objects	-	2
Precast formwork spalling	-	1
Static overload	8	-
Fatigue	3	-
Other cracks	7	-
Erosion	1	1
	39	11

Bridge Durability Parameters

Paramètres de durabilité de ponts

Brücken-Dauerhaftigkeitsparameter

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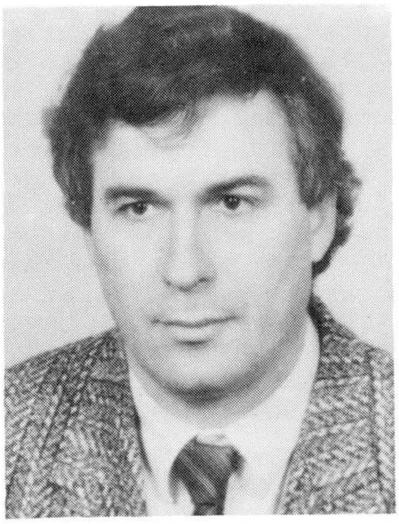

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SUMMARY

Throughout history a great many bridges have been built on Yugoslavian soil, among them many being exceptional structural and traffic achievements. Detailed data on their present state are presented in the paper, from which follows the need for regular maintenance, but also for the definition of guidelines for design and construction in especially aggressive environments.

RÉSUMÉ

A travers toute l'histoire, une multitude de ponts a été construite sur sol yougoslave. Plusieurs de ces ouvrages sont des réalisations remarquables. Dans l'article, des données précises sur leur état actuel sont présentées. Il en ressort la nécessité d'un entretien permanent ainsi que le besoin d'élaboration de directives relatives à la conception et à la construction dans un environnement agressif.

ZUSAMMENFASSUNG

Zahlreiche Brücken, davon einige mit aussergewöhnlichen konstruktiven Leistungen, wurden in der Geschichte Jugoslawiens gebaut. Daten über ihren heutigen Zustand und die Notwendigkeit der regelmässigen Unterhaltung, sowie Richtlinien für Entwurf und Ausführung für besonders aggressive Umweltbedingungen sind ausführlich dargestellt.

1. INTRODUCTION

Throughout history, paramount accomplishments in bridge construction have often been achieved on the territory of Yugoslavia. Terrain characteristics, main European traffic routes, the presence of superior builders, from Apolodoros and Mimar Sinan to contemporary ones, as well as contacts between prevailing cultures and civilizations influenced the creation of numerous bridges of great value, many of which still endure today. Particularly strong impetus in bridge construction took place during the three decades after World War II. In this period, several thousands of interesting bridges have been constructed, among them a number with exceptional dimensions and accomplishments, as is, for instance, the Tito's Bridge between land and the Krk island with the largest, 390 m span, reinforced concrete arch in the world.

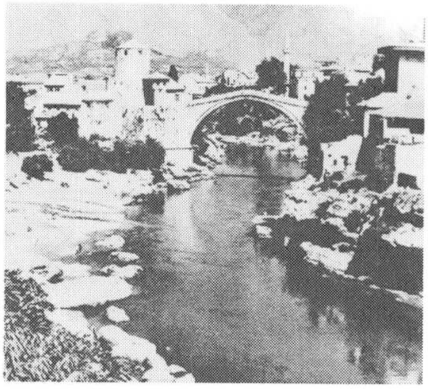

Fig 1. View of Mostar Bridge, built in XVI century

In the last several years, particular significance is being given to the bridge durability parameters analysis, by studying existing structures to determine optimal measures for enhancing their usability and durability. Also the necessary parametrical studies are carried out to investigate general data relevant for assessing the design life of structures to be designed or constructed in the future.

The first comprehensive inspection of 2210 bridges on main roads in Croatia has just been completeed. Thus, a very detailed and complex data base comprising the basic characteristics of such a large number of bridges and their present condition is being formed, on the basis of which parameters of their durability and main causes of damage are being analysed and urgent and long-term maintenance works planned.

2. DATA BASE ON BRIDGES

Modern bridge management services and the planing of all the pertaining activities calls for a comprehensive and well developed data base and an efficient working system with a prompt and regular inspection service. [1]

By the term "data base" we understand a set of all meaningful data on the initial and present properties of any particular bridge, with the programme of planned works, inspections and regular maintenance, as well as traffic and other loads imposed upon the bridge. Such a set, managed by experts and backed up by a contemporary information system and computer techniques, may at all times yield the information on:

-the levels of bearing capacity and serviceability, i.e. degree of degradation,
-the possibilities of allowing the passage of actual extraordinary loads,
-the plan, the schedule and instructions for regular maintenance work,
-specific elements associated with each particular bridge in the system,but also for groups of structures, as are data related to the planning of equipmentand means for regular and irregular works on bridges.

Therefore, the data base should contain regularly updated data:

* Main initial data on a bridge: location, disposition, structure critical details, wearable elements, design loads, construction method, interruptions during construction, built-in materijals, achieved quality, results of carried out examinations and trial loadings.
* Planned programmes of regular inspections for every bridge with directions and manner of inspection depending on the level of investigation. It would be convenient that the inspectors, during lower level inspections, be provided with forms in which they answer prepared questions.
* The plan of regular maintenance works comprising the schedule for the replacement of wearable parts and equipment, surface protection, renewal of surfacing, as well as the maintenance of special parts and equipment (e.g. on movable bridges) etc.
* Return data determined during regular and extraordinary inspections, with detailed descriptions of all observed damages, their causes, state and progress, as well as changes in the materijal. This group also comprises data on works carried out on the bridge the actual traffic conditions and irregular incidents.
* Special data on structural specifics, e.g. conduits in it, special equipment, strategic importance for the economy and other activities etc.

Besides updating these data, the bridge management system should be programmed to automatically output data on the degree of degradation, or on the threatened bridge properties, comparing them with the initial data, the data during a previous period and the allowed ones.[2]
Thus formed and updated data base presents a basis for the workout of efficijent programmes and studious projects for the maintance and protection of structures, the achievement of the planed and

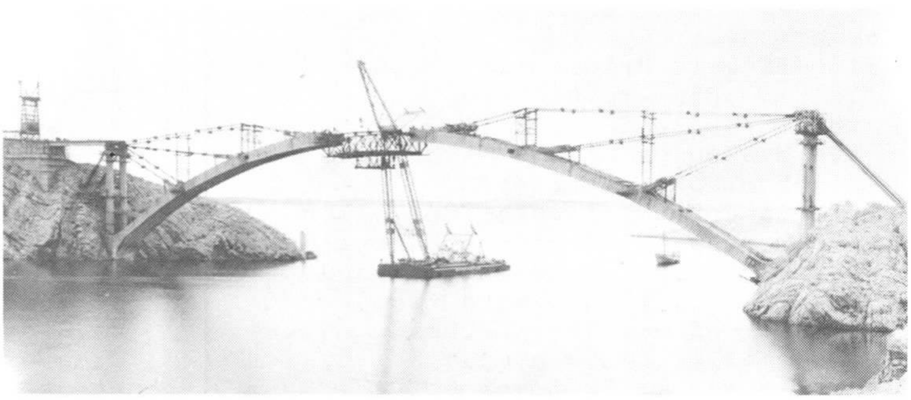

Fig 2. Reinforced concrete arch of Pag Island bridge during construction

designed durability, along with the compliance with bearability, safety and serviceability conditions.

3. INFLUENCES ON BRIDGE DURABILITY

On the basis of a comprehensive analysis and parametrical studies of safety and serviceability levels of a large number of bridges of different age the main factors of influence to the durability of bridges may be determined:

- general characteristics of traffic, particularly useful loads,
- influence of surrounding media, especially specific conditions caused by the action of sea and salt on great reinforced concrete bridges along the Adrtatic coast,
- influence of bridge characteristics (structure, materials, construction procedures, applied protection),
- influence of maintenance (from regular inspections to rehabilitation works).

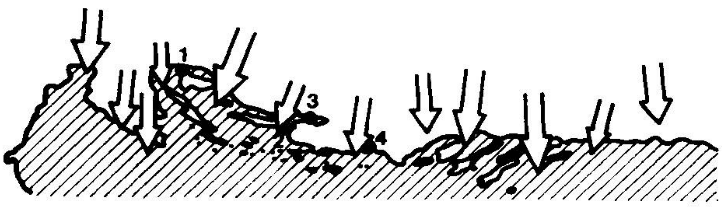

Fig 3. Locations of large arch bridges on the Adriatic coast and zones with particularly strong bura winds
Bridges: 1-Krk; 2-Pag; 3-Maslenica; 4-Sibenik

There are cases when certain, sometimes unexpected, factors lead to a significant durability decrease, and even a catastrophe, but conditions are not ancommon either when some of the mentioned factors take a dominant role and, if due measures for their neutralization are not undertaken, we may be witness unwanted consequences.

A good example of such a dominant influence on the decrease of the expected bridge durability is the aggresive action of sea water which has been observed and analysed on large Adriatic bridges for several decades already.

As known, along the coast and across channels and backwaters of the Adriatic Sea some ten very large bridges have been completed, and among them:

- the Sibenik Bridge (1966), with a 251 m span reinforced concrete arch,
- the Maslenica Bridge (1958), with a 155 m span steel arch,
- the Land-Pag Island Bridge (1968) 198 m span reinforced concrete arch,
- Tito's Bridge, between land and Krk Island (1980), with two reinforced concrete arches, the smaller a 250 m span, the larger a 390 m span.

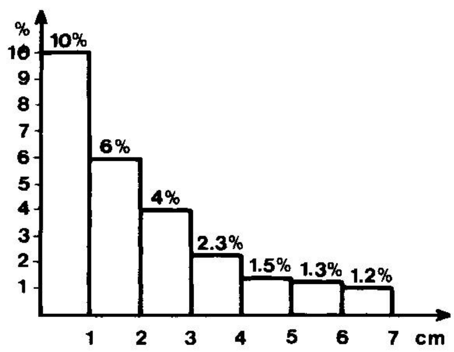

Fig 4. Diagram showing ion content compared with the quantity of cement from surface towards the interior of the Pag Bridge grider

They are exposed to an extremely aggresive action of sea water. The majority of them are located in zones with very strong bura winds, as is the Pag Bridge, where during its construction in 1967 builders recorded 241 day with force 6 winds or stronger, of which 138 days with force 8 winds or stronger. The bura is characterized by very cold winds which blow down the ridges of littoral mountains reaching greatest force along the sea surface, lifting sea drops and soaking the surroundings and these bridges above the sea.

The Adriatic Sea water has approximately the following content of aggresive ions:

SO_4^{2-}	2970 mg/l	Cl^-	21250 mg/l
Mg^{2+}	1420 mg/l	Na^+	11810 mg/l
Ca^{2+}	457 mg/l	K^+	390 mg/l

A detailed analysis and experimental investigation carried out on parts of Pag Bridge has shown that the cause of this aggresive action of the surrounding environment is best observed by the presence of free chlorides in the concrete of the bridge's pavement structure main girders. The amount of chlorides in the surface layer of concrete (up to a 1 cm depth).
In the surface layer (up to 1 cm thick) of concrete the quantity of chlorides compared to the quantity of built-in concrete is 10% and, as shown on Fig. 4., even at the depth of 7 cm (where cables are placed) the chloride content is 1.2% . All this Significantly exceeds allowed levels (e.g. according to Page [3]) even those classified as a great risk of reinforcement corrosion.[4]

The consequence of this is the significant reinforcement corrosion owing to which remedial works are carried out to upgrade the safety and serviceability of the bridge.
Sae water aggressiveness causes not only reinforcement corrosion but also concrete quality decrease, degradation of dilatations, bearings, fences etc and, in cases of metal bridges, the deterioration of every bridge part which is not maintained regularly.

The efects of these aggressive actions on durability have not been known completely and in all their manifestations during the construction of these bridges, but on the basis of actual data on their behaviour guidelines are being worked out as a foundation for the design and construction of structures along the future Adriatic Motorway and other structures in the region also

4. REWIEW OF CONDITION AND DAMAGE CLASSIFICATION OF BRIDGES IN YUGOSLAVIA

During the past five years all most important bridges in Yugoslavia have been inspected with the aim of forming a base of data and planning of the mcst urgent maintenance works on them. Data on the condition these structures are in are illustrated on the example of SR Croatia, where a total of 2210 bridges has been inspected:
- in good condition..........................25.16 %
- with minor deficiences.....................56.38 %
- with major deficiences.....................12.58 %
- worn out................................ 5.88 %

On the inspected bridges frequency of defects is different on various parts of structures. Namely, on the total number of inspected bridges defects have been observed in following percentages on:
- fences and bumpers..........................59 %
- dilatations.................................50 %
- gutters.....................................47 %
- abutments...................................35 %
- span structures.............................31 %

and on the remaining parts of the structures in lesser percentages.

This already points to the fact that defects are mostly a consequence of deficiencies in design, construction or maintenance and sometimes, inordinate traffic action. a precise analysis of all deficiencies has determined that they are a consequence of:
- design ....27 %
- construction ....41 %
- lack of maintenance.........................32 %
- other causes ....10 %

But this distribution should not be accepted anconditionally, as many deficiencies have such a character that, although they are a consequence of faulty design or construction, they can be eliminated or significantly reduced with adquate maintenance. Also a minor fault in the stage of design or during construction may acquire significant dimensions due to inadequate maintenance.

5. CONCLUSION

From the presented information on the relatively inferior condition and significantly reduced serviceability of a large number of anaysed data the following ensues:
- undelayable need for forming accurate data bases on bridge condition in every transportation system,
- timely and urgent maintenance and damage remedial works at the earliest stages of their manifestation,
- definition of criteria for the design and construction of structures in particularly aggresive environments or similar extraordinary conditions.

All these activities are currently being carried out on the system of bridges in the Yugoslav road network with the goal of regaining the high standard and exceptional accomplichments achieved by their construction.

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Durabilité de l'état de surface des bétons d'équipements urbains

Dauerhaftigkeit der Betonoberflächen von städtischen Betonbauten

Durability of Concrete Surface of Concrete Public Works

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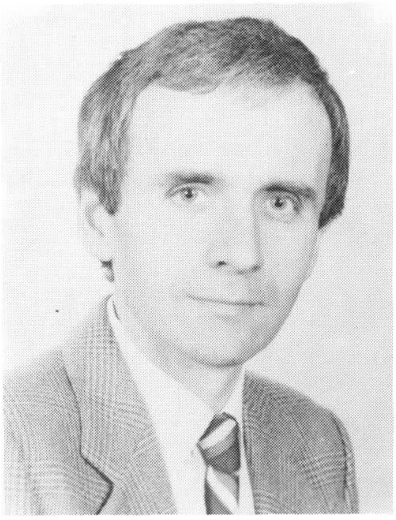

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RÉSUMÉ

Le vieillissement des parements de béton des ouvrages publics équipant les villes est loin d'être inéluctable. L'article analyse des causes de dégradations et les moyens de les prévenir.

ZUSAMMENFASSUNG

Die Alterung der Betonaussenseiten von öffentlichen Bauwerken in Städten ist nicht unvermeidlich. Der Bericht untersucht die Gründe der Schäden und die Vorbeugungsmassnahmen.

SUMMARY

The ageing of the surface of concrete public works is far from being unavoidable. This article analyses the causes of damage and the ways to prevent them.

1. LES OUVRAGES EN BETON

- Le béton occupe une place de plus en plus importante dans les ouvrages publics équipant nos villes ; ce, sous des formes multiples, plus ou moins volumineuses et marquantes.
- Au premier plan se situent, bien entendu, les ouvrages d'art tels que viaducs, ponts et grands murs de soutènement.
- Mais il y a aussi les bâtiments publics - cités administratives - hôpitaux - établissements scolaires, qui font un large appel au béton apparent, architecnique en règle général, et, à une moindre échelle, les composants des aménagements et mobiliers urbains.
- En règle générale ces ouvrages ne sont pas "habillés", le béton brut de décoffrage restant apparent. Parfois le béton reçoit une finition, du genre bouchardage ou bien une peinture, mais ces traitements peu fréquents relèvent plutôt du "cache-misère" pour un état de surface peu satisfaisant.

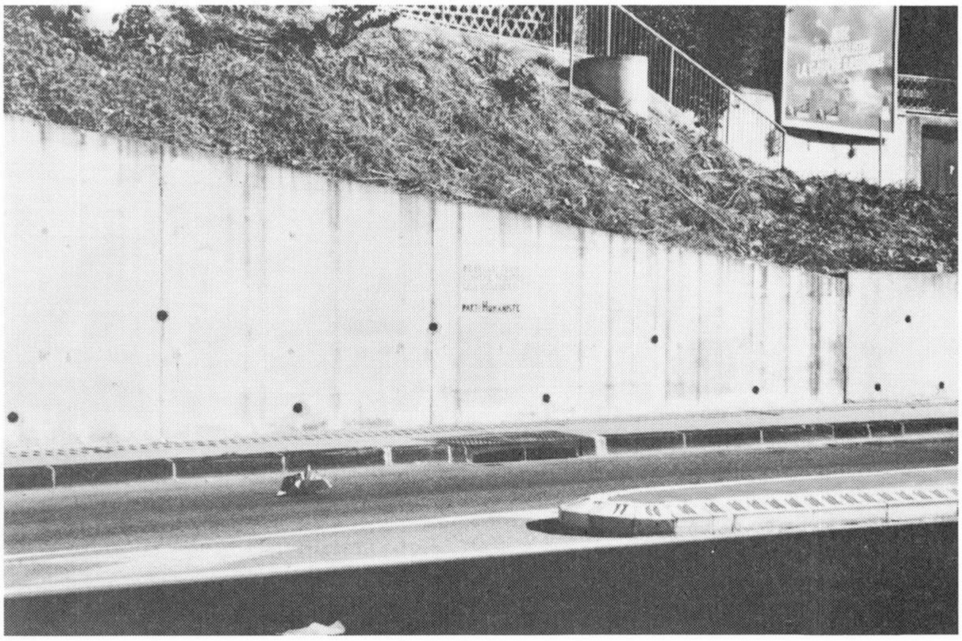

Fig.1 Béton brut sans précautions

2. LES AGRESSIONS SUBIES PAR LES PAREMENTS DU BETON

- Les agressions sont nombreuses et variées. Suivant leur nature elles entraînent des dégradations allant du simple salissement à la corrosion de surface plus ou moins profonde et plus ou moins accompagnée de lichens et autres mousses entretenant le processus de dégradation.
- Au premier rang, en milieu urbain, se situent les agressions athmosphériques, fumées des foyers domestiques et industriels, gaz d'échapement des voitures et, suivant la situation géographique, air salin. Ces fumées et gaz d'échappement, fortement chargés en gaz carbonique et dérivés de souffre ont une action chimique directe sur le béton.
- Vient ensuite la poussière. En milieu urbain les poussières sont fortement chargées en silicates (40 %), carbonate (20 %), matières organiques (10 %), sels

solubles (10 %), graisses (10 %), suie (5 %), et humidité (5 %). Ce cocktail adhère par tension superficielle à la surface du béton lorsque, et c'est presque toujours le cas, l'humidité de l'air dépasse 50 %. La perméabilité de surface du béton, même faible, les microfissurations et les faïençages permettent la pénétration des poussières dans l'épaisseur du parement, pénétration accélérée par les eaux de pluies ruisselantes. Alors le parement noircit progressivement avec une intensité plus marquée sur les cheminements préférenciels des eaux de ruissellement. A un stade avancé, lichens et mousses s'installent "mangeant" peu à peu la surface du béton.

- Concernant les cheminements des eaux de ruissellement, il importe de préciser que, dans la majorité des cas, ils sont la conséquence d'une mauvaise conception des formes architecturales de l'ouvrage ou du parement lui-même, comme, par exemple, les "moustaches" des appuis de fenêtres ou d'éléments architectoniques saillants.
- La pluie, déjà active dans le processus précédent, a également une part directe dans la dégradation des parements lorsqu'elle est chargée en acides, provenant des fumées industrielles essentiellement.
- Autre agression, humaine celle-ci, mais importante, les graffiti et affiches. Les grandes surfaces planes des parements béton sont malheureusement un support idéal pour ceux-ci et rares sont les ouvrages qui y échappent, au moins à hauteur d'homme, mais cela va parfois au-delà et la malignité humaine est alors sans limites.
- Restent, à une moindre échelle, l'abrasion par le vent chargé de sable qui, en certains sites très exposés, a des effets nettement perceptibles et les agressions mécaniques, volontaires ou non, surtout sensibles sur les arêtes des ouvrages.

3 LES MOYENS A METTRE EN OEUVRE CONTRE CES AGRESSIONS

- Pour les agressions mécaniques, on conçoit bien que les protections ne peuvent être qu'extérieures à l'ouvrage lui-même et, à l'instar des bornes flanquant les anciennes portes cochères, constituer des "pare-chocs", en général métalliques, avancés et interdisant le contact direct. En leur absence, il n'y a plus d'autres recours, après choc, qu'en la réfection.
  Il existe par contre de nombreuses techniques préventives contre les autres agressions, sinon à effet définitif du moins garantissant une durabilité très nettement supérieure et des conditions d'entretien faciles.

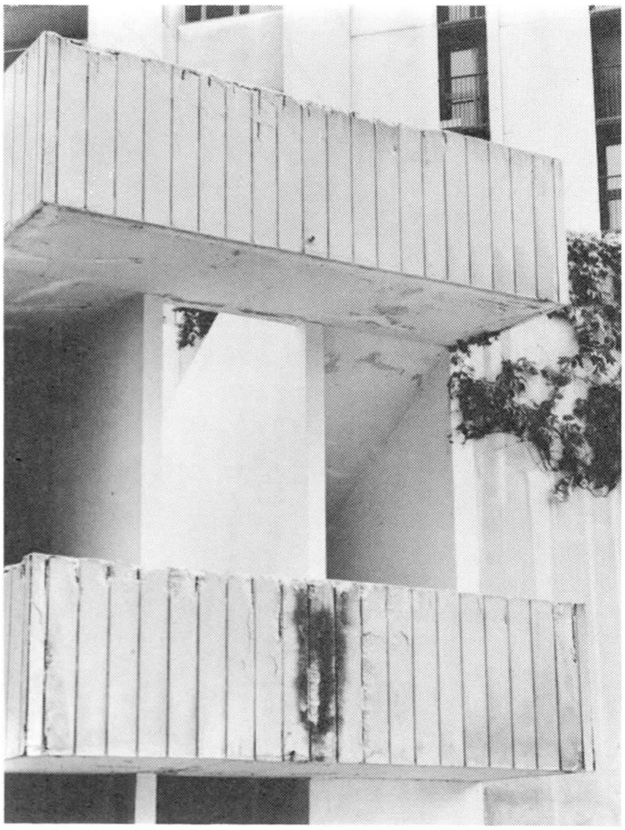

Fig.2 Béton mal traité

3.1 Qualité du béton

- La première gamme de techniques concerne le béton lui-même, tant dans sa confection et sa mise en oeuvre que dans sa conception architectonique.
- Une bonne conception architectonique doit être guidée par le soucis d'éliminer, autant que faire se peut, les causes de ruissellements systématiques, éléments saillants en particulier, ou de rétention d'eau et aussi les "masques" aux vents dominants créant des zônes jamais lavées par la pluie.
- La qualification initiale des bétons et de leur mise en oeuvre est évidemment une condition essentielle à remplir pour obtenir une bonne résistance aux agressions.

  Cette qualification porte sur :

- La composition du béton lui-même, le choix du ciment (p.ex.béton blanc), et des agrégats, dont la coloration a un rôle important quant à la couleur finale du parement et à la stabilité de celle-ci.
- L'incorporation éventuelle de colorants (pigments naturels minéraux).
- les dosages précis et constants ainsi que l'homogénéité et la densité, facteur essentiel de la durabilité, obtenues par un malaxage optimum.
- La conception des coffrages, d'une part quant à leur calepinage lui-même et à la répartition des trous de coffrage qu'il est souhaitable en outre d'équiper de cols propres et de ne pas reboucher ; d'autre part, quant à leur mise en oeuvre précise. Enfin un soin particulier doit être attaché à l'application des produits de décoffrage qui doivent être propres, sans souillures et absolument neutres à l'égard du béton.
- Le bon positionnement des armatures, évitant des fers trop proches de la surface et créant des points de faiblesse du parement.
- Une mise en oeuvre sérieuse par couches horizontales relativement minces, avec compactage à l'aiguille poussé pour obtenir un béton très homogène, très dense et sans lignes de faiblesse en parement.
- Un bon durcissement du béton, bien contrôlé, aidé au besoin par des adjuvants adaptés.

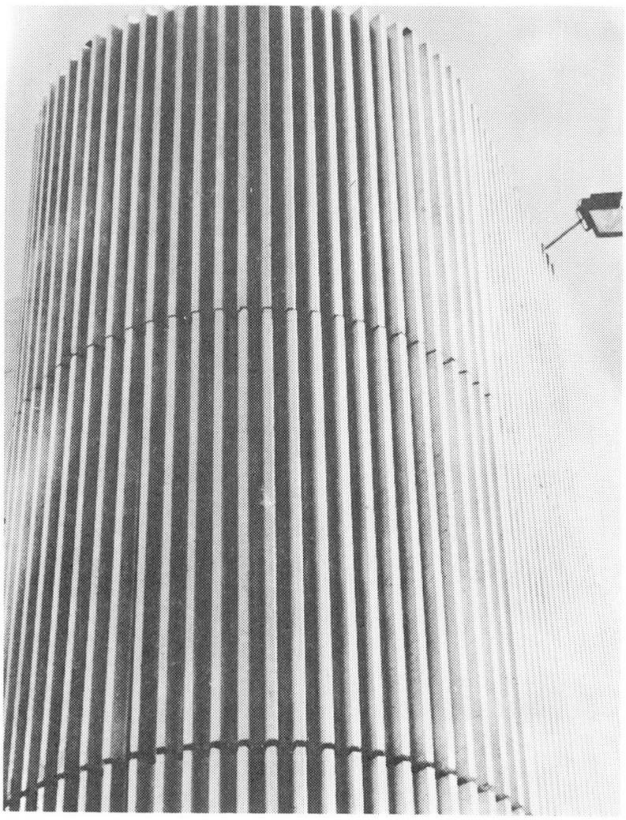

Fig.3 Béton urbain

3.2 Traitement des surfaces

Ensuite viennent les traitements des surfaces dont les objectifs sont:

- soit de renforcer la dureté de surface et d'éliminer la porosité et le faïençage du parement - sources de pénétration de l'eau et des poussières, tels le lavage du parement avec un fluosilicate de magnésium - trois semaines après la coulée - transformant les carbonates de chaux en fluorures de chaux, sels durs et insolubles, résistants à l'érosion, imperméables et résistant aux attaques chimiques.

- soit d'imperméabiliser le parement, interdisant ainsi le développement des mousses et des lichens et facilitant l'auto-élimination des salissures hydrosolubles et le nettoyage des parements.

Les produits d'imperméabilisation agissent par pénétration dans le support tout en le laissant respirer.

Ces produits, pour être efficaces, doivent avoir une très faible tension superficielle assurant une pénétration profonde inférieure à 9 mm, avoir une bonne résistance à l'oxydation, à l'alcalinité du béton et à l'action des rayons ultra-violets.

Doivent être évités tous les produits aboutissant à la formation d'un film homogène non pénétrant dont l'adhérence au support n'est pas bonne et qui peuvent cloquer puis se désquamer.

De nombreux produits existent commercialement, essentiellement à base de silicones ; encore faut-il être très sélectif quant à leur formulation portant en particulier sur leur fluidité et leur pouvoir de pénétration, condition essentielle d'un bon résultat. En effet une pénétration lente se traduit par la formation en surface d'une "peau" non stabilisée et qui, par collage ou effet électrostatique, fixe les poussières, aboutissant à l'inverse du résultat souhaité.

Il existe, moins répandues sur le marché, d'autres résines plus performantes, à base d'oligomères de siloxane, très fluides et ayant donc un fort pouvoir de pénétration, qui ont la particularité d'une polymérisation lente, se terminant donc dans la masse du béton et dormant une excellente homogénéité du produit avec celui-ci.

En règle générale les meilleurs produits peuvent donner une garantie décennale mais il n'en demeure pas moins qu'alors que les traitements de durcissement sont définitifs ceux d'hydrofugation sont à renouveler périodiquement.

3.3 Protection des parements

Enfin il y a la protection des parements contre les graffiti et les affiches. Cette protection a un but essentiellement esthétique, graffitis et affiches ne remettant pas en cause la stabilité chimique et mécanique du béton. En outre, le fait de traiter les surfaces ne dispense-t-il pas d'un suivi régulier des ouvrages et d'interventions rapides de nettoyages et de renouvellement de la protection.

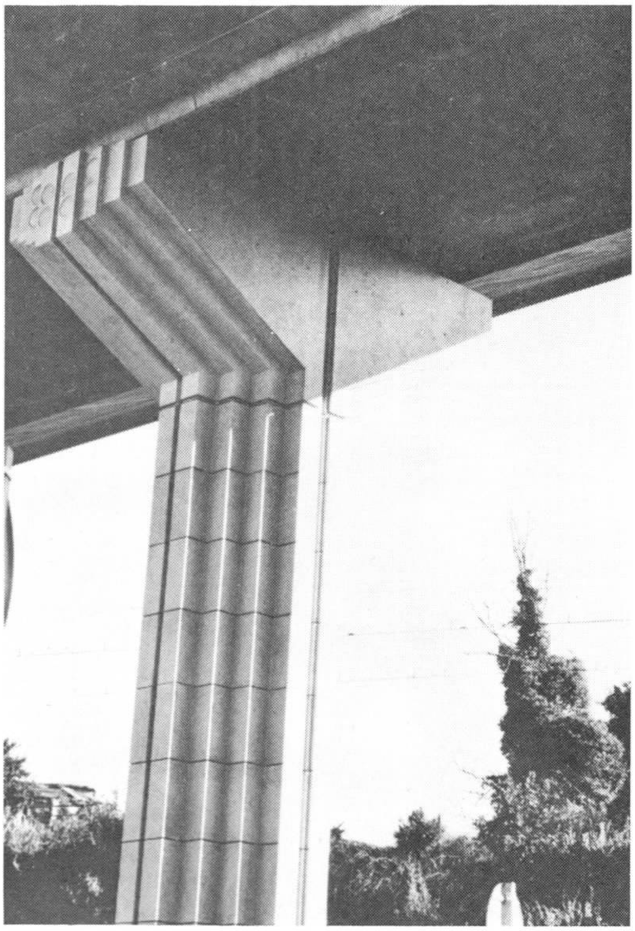

Fig.4 Pile soignée

- En effet, les résines employées empêchent effectivement la pénétration des peintures et colles jusqu'au béton mais en fixent une partie, et d'autant plus que l'intervenon de nettoyage est tardive. Pour le nettoyage il est donc nécessaire d'employer des diluants compatibles avec les résines et on aboutit à une perte de matière qui doit immédiatement être compensée par une nouvelle application.

- Il est à noter que, hydrofuges on antigraffitis, tous ces produits ont un très faible pouvoir colorant et ne modifient qu'imperceptiblement la teinte d'origine du béton. Mais ils peuvent également être colorés - essentiellement avec des pigments minéraux - si l'on recherche un effet décoratif sans, pour autant, vouloir "peindre" le parement.

4. CONCLUSION

Ainsi, comme nous venons de le voir, le vieillissement des parements de béton est loin d'être un phénomène inéluctable.

Encore faut-il que Maîtres d'Ouvrages, Maîtres d'Oeuvres et Entreprises aient pleinement conscience de l'enjeu, tant sur le plan de l'esthétique que sur celui de la maintenance des ouvrages et sachent accepter pleinement les contraintes techniques et financières seules susceptibles de garantir la beauté et la durabilité des ouvrages d'art en béton.

Grundsätzliche Anmerkungen zur Dauerhaftigkeit von Stahlhallen

Basic Remarks on the Durability of Single-Storey Steel Buildings

Remarques fondamentales sur la durabilité des halles métalliques

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Joachim Scheer, geb. 1927, promovierte 1958. Als Beratender Ingenieur befasste er sich mit der Planung von Brücken, Masten und Gerüsten. Seit 1971 zunächst in Hannover, ab 1976 in Braunschweig Professor für Stahlbau, arbeitet er vorwiegend an Problemen der Plattenstabilität, der Berechnung von Masten und der Normenentwicklung.

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Hartmut Pasternak, geb 1954, promovierte 1981. Danach befasste er sich mit Problemen des Stahlhallen-, Kranbahn- und Silobaus. Seit 1986 ist er wissenschaftlicher Mitarbeiter am Institut für Stahlbau in Braunschweig und arbeitet an Fragen der Grenztragfähigkeit von Trägern und der Lastmodellierung.

ZUSAMMENFASSUNG

Ausgewertet werden zunächst über 100 mittlere und grosse Schadenfälle von Stahlhallen. Anhand ausgewählter Beispiele werden dann qualitative und quantitative Aspekte heutiger Gebrauchstauglichkeitskriterien im Hinblick auf den tatsächlichen Erhalt der Funktionstüchtigkeit untersucht.

SUMMARY

Evaluations are made for more than 100 middle-sized and large cases involving damage to single-storey steel buildings. On the basis of selected examples, qualitative and quantitative aspects of current serviceability criteria in regard to the real maintenance of the utility are investigated.

RÉSUMÉ

Plus de cent cas de dommages de moyenne et grande importance de halles métalliques ont été étudiés. Des aspects qualitatifs et quantitatifs des critères de l'aptitude au service ont été déterminés à l'aide de cas choisis, pour vérifier les besoins d'entretien.

1. STAHLHALLEN UND DER BEGRIFF DAUERHAFTIGKEIT

Stahlhallen gehören in den Industrieländern seit Jahrzehnten zu den Standarderzeugnissen der Baubranche; in Großbritannien werden ca. 90% aller Hallen in Stahl errichtet, in Italien sind es nur etwa 10%; die BR Deutschland liegt mit ca. 40% im Mittelfeld.

Trotzdem hat es bisher - auch in dieser Hinsicht sind Hallen typische Bauerzeugnisse - keine systematischen Schadensanalysen wie etwa in der Automobilindustrie gegeben, die Aufschluß über die tatsächliche Dauerhaftigkeit solcher Objekte geben könnten.

Unter Dauerhaftigkeit eines Bauwerks versteht man seine Fähigkeit, sich während seiner Lebensdauer funktions- und betriebsgerecht zu verhalten. Die Deutung des Begriffs Dauerhaftigkeit ist unter Bauingenieuren nicht unumstritten. Im Entwurf der SIA 160 (März 1988) ist Dauerhaftigkeit nur eine Teilmenge der Gebrauchstauglichkeit; im Entwurf des EC3 (Juli 1988) steht sie eher neben Tragfähigkeit und Gebrauchstauglichkeit, wenn u.a. gefordert wird,

- Umwelteinfüsse und Bauwerkslage,
- Nutzung und erwartete Lebensdauer des Bauwerks,
- Eigenschaften der verwendeten Werkstoffe,
- Gestaltung der Bauteile und konstruktiven Details,
- Qualität der Ausführung und Niveau der Qualitätskontrolle sowie
- eine geeignete Bauwerkserhaltung

bei der Bemessung zu berücksichtigen. Schließen wir uns der zweiten Auffassung an und werfen wir einen Blick auf die Grenzzustände von Bauwerken.

Heutige Regelwerke - wie etwa der Entwurf des EC 3 - gehen aus von einer Lebensdauer von 50 Jahren und begrenzen für diesen die rechnerische Versagenswahrscheinlichkeit auf

- p_f = 5·10E-5 für die Grenzzustände der Tragfähigkeit,
- p_f = 5·10E-2 für die Grenzzustände der Gebrauchstauglichkeit.

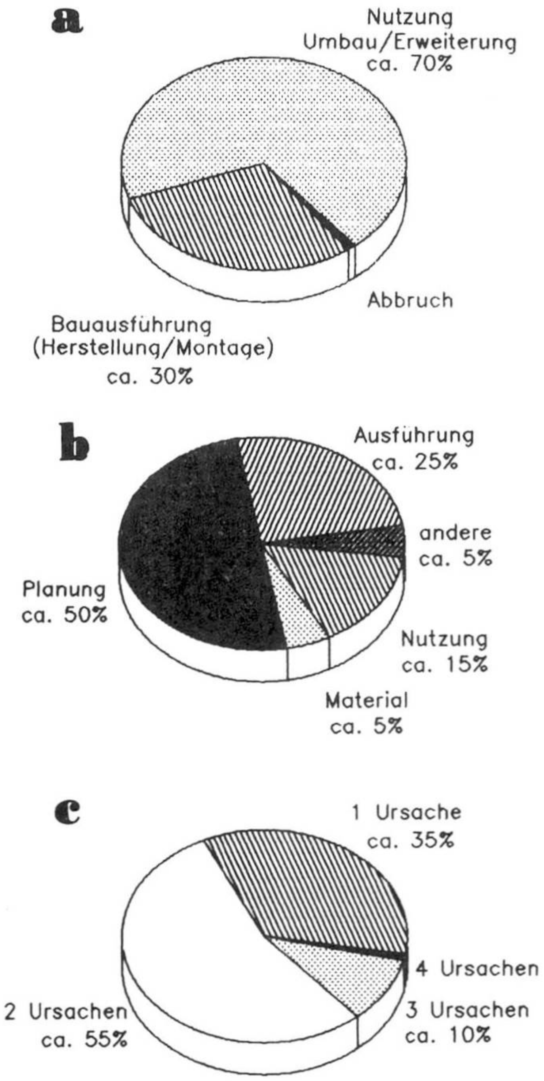
Dem stehen nach unserer Kenntnis folgende Orientierungswerte für Hallen gegenüber:

- für die jährliche Häufigkeit von Katastrophen mit großem finanziellen Verlust oder Personenschaden: $2*10^{-5}$,
- für die Häufigkeit von wesentlichen Nachbesserungen zur Gewährleistung der Gebrauchstauglichkeit: 1 mal pro 50 bis 100 Hallen.

Durch die in der probabilistischen Analyse nicht erfaßten Einflüsse (Fehler bei der Herstellung und Nutzung) wächst die Versagenswahrscheinlichkeit ausgeführter Tragwerke üblicherweise auf ein Vielfaches. So erscheint hier die Differenz in den Versagenswahrscheinlichkeiten im Grenzzustand der Tragfähigkeit eher gering, was auf "Reserven" der konventionellen Bemessung (etwa von Rahmen oder durch Vernachlässigung der räumlichen Tragwirkung) schließen läßt. Bedenken läßt dagegen der gravierende Unterschied bei den Versagenswahrscheinlichkeiten in den Grenzzuständen der Gebrauchstauglichkeit aufkommen.

2. SCHÄDEN - AUSWERTUNG EINER STICHPROBE

Wenn systematische Schadensanalysen nicht geführt werden oder werden können, ist die Untersuchung von Stichproben meist zweckmäßig. Nachfolgend wird über die Ergebnisse einer Auswertung von über 100 mittleren (d.h. mit erheblichem Aufwand sanierbar oder Personengefährdung) bis großen (d.h. großer finanzieller Verlust oder Personenschaden), in den vergangenen 20 Jahren bekannt gewordenen internationalen Schadensfällen an Stahlhallen berichtet. Kleinere Schäden, die den Bereich der öffentlichen Sicherheit nicht berühren, sind in der Literatur nicht dokumentiert. Die Untersuchung zeigt, wann (Bild 1a) bzw. durch welche Fehler (Bild 1b) diese Schäden auftraten - es dominieren die Nutzungsphase bzw. Planungsfehler. Zu letzteren gehören mehrfach nicht zutreffende Schneelastannahmen (teilweise wegen unzureichender Normwerte) und subjektive Fehler wie z.B. Nichtverfolgung von Schnittkräften bis in Verbindungen. Nutzungsfehler treten in erster Linie auf infolge Beanspruchung durch Lasten, die im Entwurf gar nicht vorgesehen waren. Auf Bild 1c ist die prozentuale Häufigkeit der Anzahl der Schadensursachen nach Bild 1b gezeigt; zwei Drittel aller Schäden sind auf eine Fehlerkombination (z.B. aus Planungs- und Ausführungsfehlern) zurückzuführen.



<u>Bild 1</u> Schadensauswertung

Deutlich wird erneut die Eigenart von Stahlkonstruktionen: dem problemlosen Baustoff Stahl stehen hohe Anforderungen an Planung (z. B. wirken sich durch den geringen Eigenlastanteil unerwartet hohe Schneelasten stärker aus), Ausführung und Nutzung gegenüber. Mittlere und große Bauschäden von Hallen werden heute nur noch selten durch neuartige technologische Probleme (wie früher z.B. durch nicht beherrschte Stabilitätsfragen oder Schweißmängel) verursacht, sondern eher durch unzureichende Koordination der am Bau Beteiligten. Das Spektrum der Schäden ist breit. Es reicht vom Einsturz ganzer Stützen-Binder-Systeme bzw. Rahmen über das Versagen einzelner Anschlüsse bis zu Verformungen der Längsverbände infolge fehlender Temperaturfuge in Hallenlängsrichtung sowie Risse in der Bauwerkshülle durch ungleiche Stützensetzungen oder fehlende Übertragung von Bremskräften aus Kranbetrieb.

Die bisherigen Untersuchungen bestätigen für Stahlhallen die Gül-

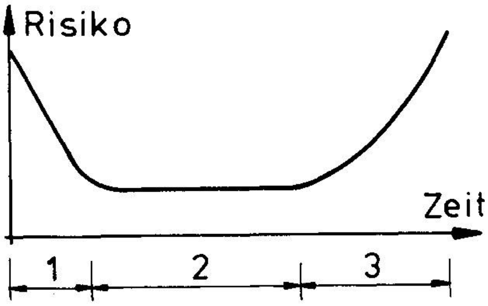


Bild 2 Risiko-Zeit-Funktion

tigkeit zumindestens der ersten beiden Abschnitte der klassischen Risiko – Zeit – Funktion (Bild 2); da Hallen keiner perfekten Kontrolle bei der Abnahme unterliegen, setzt mit der Nutzung die natürliche Selektion (1) ein; das verdeutlichen z.B. die Schneelasten: Schäden unter dieser Last sind während der ersten fünf Nutzungsjahre viermal häufiger als in den verbleibenden. Weitere Phasen sind die reguläre Nutzung (2) und das Altern (3), über dessen Beginn und Verlauf mit dieser Stichprobe keine ausreichend gesicherten Erkenntnisse gewonnen werden konnten.

Inwieweit ist die Untersuchung mittlerer und großer Schadensfälle für Dauerhaftigkeitsbetrachtungen relevant? Eine Antwort gibt Bild 3. Es zeigt anhand des nachfolgenden (sicher extremen) Beispiels, daß Fehler, die in dieser Auswertung zum Verlust der Tragsicherheit führten, unter anderen Umständen "nur" eingeschränkte Gebrauchstauglichkeit nach sich gezogen hätten.

Bei starker Schnee- und Windbelastung sowie einer Temperatur von -22°C versagten die Schrauben, mit denen das Zugband eines unterspannten Fachwerkriegels (Spannweite 15 m) an dessen Knotenblech befestigt war. Das führte zum Versagen der (unten eingespannten) Stützen im Unterbereich bzw. zu starken Abweichungen aus dem Lot. Während im Entwurf ein verschiebliches Lager für den Fachwerkbinder vorgesehen war, wurden beide Lager unverschieblich ausgeführt. Das ergab zusätzliche Kräfte aus Wind, Temperatur und unsachgemäß gelagertem Lagergut. Hinzu kam die unzureichende Ausführung mehrerer Schweißnähte. Insgesamt stürzten 10 Riegel ein.

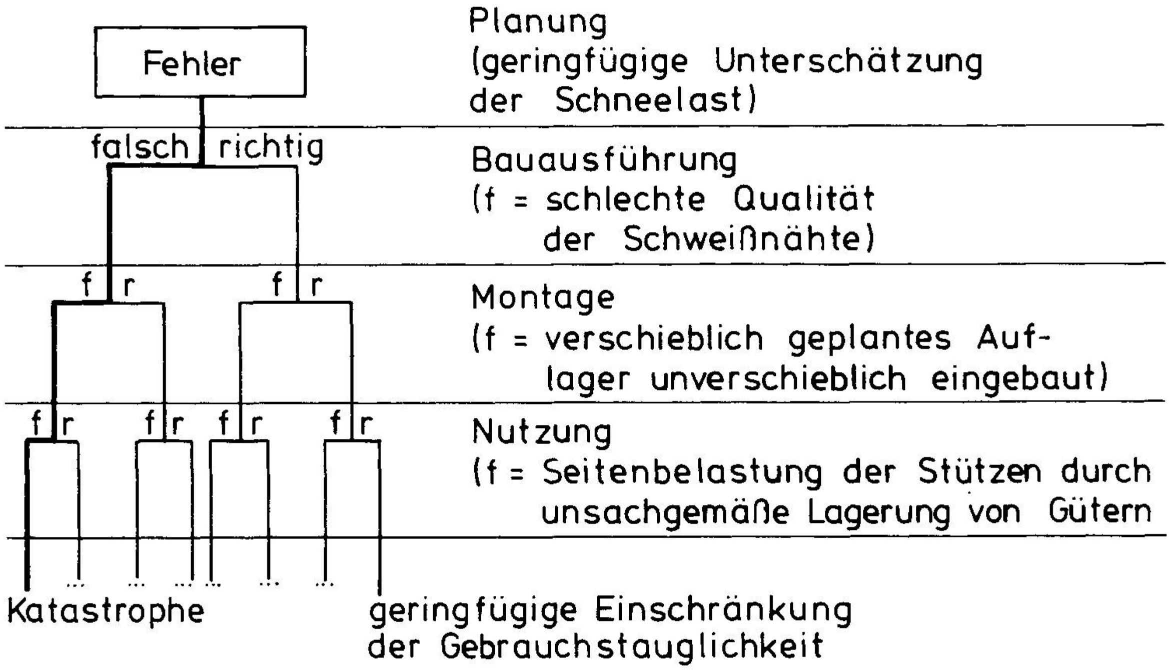


Bild 3 Ereignisbaum Stützen-Binder-System

Nicht erfaßt werden konnten mit dieser Stichprobe möglicherweise signifikante Unterschiede zwischen Massenherstellung und individueller Anfertigung, zwischen Industrie- und Ausstellungshalle. Bestätigt wurde dagegen, daß sich Nutzungsziele nicht immer durch Berechnungen erzielen lassen, sondern auch durch die Wahl geeigneter konstruktiver Durchbildung, durch Ausführung, Kontrolle und Unterhalt. Nachfolgend widmen wir uns der rechnerischen Seite.

3. ÜBER GEBRAUCHSTAUGLICHKEITSKRITERIEN

3.1 Qualitative Aspekte

Auch hier wieder ein Beispiel. Über die Dauerhaftigkeit einer Kranbahn entscheiden bekanntlich insbesondere die Schiefstellung der Kranlaufräder sowie dauerhafte und elastische Deformationen in der Horizontalebene der Fahrbahn.

Heutzutage beschränken wir uns beim sog. Gebrauchstauglichkeitsnachweis allein auf die Begrenzung der elastischen Durchbiegung der einzelnen Kranschiene auf z.B. 1/800, der Kopfauslenkung der einzelnen Kranbahnstütze auf h/300 bzw. der Differenz der Kopfauslenkung gegenüberliegender Stützen auf 20 mm, fordern also eine Art Mindeststeifigkeit. Nun ist z.B. die zweite dieser Forderungen berechtigt, etwa um das Herausspringen von Fensterscheiben beim Vorbeifahren eines Brückenkrans (ein Schadensfall in Abschnitt 2) zu verhindern. Was aber bedeuten diese Kriterien für den "Leitgebrauch"?

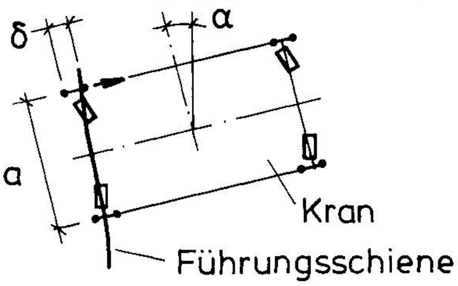


Bild 4 Kranschrägstellung

Zum Nachweis der Gebrauchstauglichkeit ist es erforderlich zu überprüfen, ob die Gesamtschrägstellung α des Krans (aus den für den Betrieb prognostizierten geometrischen Imperfektionen und elastischen Verformungen der Kranbahn in der Horizontalebene einerseits sowie infolge prognostizierter Laufradschrägstellung andererseits) kleiner ist als die überhaupt mögliche Schrägstellung, die sich als Quotient aus dem Spurspiel δ zwischen Führungselement und Schiene sowie dem Abstand a zwischen den Führungselementen ergibt (Bild 4). Ist diese Bedingung erfüllt, fährt der Kran im sog. hinteren Freilauf - einer Stellung, die sich günstig auf die Dauerhaftigkeit der Kranbahn auswirkt.

Wir erkennen an diesem wiederum drastischen Beispiel, daß Vorschriften zwar Schritte zur Sicherung der Dauerhaftigkeit vorsehen, ohne daß dann daraus später immer Schlußfolgerungen hinsichtlich der Dauerhaftigkeit gezogen werden können. Neue Gebrauchstauglichkeitskriterien müssen gefunden werden.

3.2 Quantitative Aspekte

Versagenswahrscheinlichkeit. In der letzten Zeit wird vorgeschlagen, in vielen Fällen größere rechnerische Versagenswahrscheinlichkeiten in den Grenzzuständen der Gebrauchstauglichkeit zu akzeptieren, z.B. p_f = 0.1...0.2/Jahr [1]. Aus dem Vergleich mit Abschnitt 1 sehen wir, daß diese Auffassung dem status quo im

Hallenbau näher kommt als der bisherige Wert p_f = 10E-2/50 Jahre (insbesondere, wenn man bedenkt, daß dort nur Schäden mit wesentlichen Einschränkungen der Gebrauchstauglichkeit erfaßt wurden).

Lebensdauer und Nutzungsbedingungen. Hüllelemente werden aus Gründen der Alterung aber auch des veränderten Geschmacks mitunter schon nach weniger als 20 Jahren ersetzt. Auch viele Brückenkrane überschreiten dieses Alter nicht: mal werden sie durch neue, leichtere Krane gleicher Tragfähigkeit ersetzt, mal werden Krane größerer Tragfähigkeit gewünscht. Das zieht veränderte Anforderungen an die Kranbahn nach sich. Ähnliches gilt für Installationen und Beleuchtung. Aber selbst grundlegende Änderungen der Nutzung von Hallen sind häufig. Der heutige, generell 50jährige Bezugszeitraum für Gebrauchstauglichkeitskriterien erscheint unrealistisch, individuelle Nutzungsprognosen sind erforderlich. Mit nur vage definierten Eingangsdaten ist eine sicherheitstheoretisch fundierte, quantitative Aussage über die Gewährleistung der Gebrauchstauglichkeit nicht möglich. Ein erster Schritt aus dieser Situation könnte sein, daß zukünftige Regelwerke verfeinerte Lastannahmen für unterschiedliche Bezugszeiträume anbieten.

3.3 Kontrolle

Kehren wir noch einmal zurück zum Beispiel aus Abschnitt 3.1. Die Ausgangsdaten für den dortigen Gebrauchstauglichkeitsnachweis, die Lage der Kranbahn und die Stellung der Kranlaufräder, werden bei der Montage vermessen und mit Herstellungstoleranzen verglichen, spätere Vermessungen von Kran und Kranbahn erfolgen meist nicht mehr regelmäßig (deren Interpretation wäre übrigens nicht problemlos, da es bis heute keine Festlegungen für Betriebstoleranzen gibt!), sondern vielfach erst dann, wenn Schadensfälle z.B. an der Kranbahn eintreten (wie in der o.g. Auswertung mehrmals registriert). In [2] wird gezeigt, wie zu verschiedenen Zeitpunkten durchgeführte Messungen der Kranbahnlage auf probabilistischer Grundlage ("Markov-Ketten") ausgewertet werden können und zu zuverlässigen Aussagen über die Dauerhaftigkeit führen. Ebenfalls abgeleitet werden kann so die notwendige Häufigkeit von Messungen, die zur "Vorwarnung" erforderlich ist. Möglich ist auch eine Zustandsprognose (mit welcher Wahrscheinlichkeit treten welche Imperfektionen auf?), wenn Ausgangssituation und Betriebsbedingungen bekannt sind.

4. SCHLUSSBEMERKUNG

Die Auseinandersetzung um das Konzept der Teilsicherheitsbeiwerte wurde angetreten mit der Absicht, die rechnerische der tatsächlichen Sicherheit anzunähern. Soll die für Stahlhallen bestehende Diskrepanz überwunden werden, muß insbesondere der Katalog der Gebrauchstauglichkeitsanforderungen (z.B. in SIA 160 und EC3-Entwurf) überprüft, neu gestaltet und stetig aktualisiert werden.

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Dauerhafte Dachkonstruktionen besonderer Industriebauten

Durable Roof Construction of Industrial Buildings

Construction durable de toits de bâtiments industriels

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ZUSAMMENFASSUNG

Auch bei modernen Industriebauten mit kritischem Chemismus, der auf das Hüllsystem des Gebäudes, insbesondere auf das Dach einwirken kann, bewähren sich häufig traditionelle Bauformen, Konstruktionen und Baustoffe — wie bereits zu Beginn des Industriezeitalters.

SUMMARY

Modern industrial buildings often involve chemical conditions which attack the shell system of the structure, especially the roof. Here traditional types of buildings, constructions and materials prove useful — just as at the beginning of the industrial age.

RÉSUMÉ

Dans de nombreux bâtiments industriels modernes règnent des conditions chimiques qui ont une influence sur les système de construction, particulièrement sur les toitures. Ici, les formes traditionelles de constructions et de matériaux font leurs preuves, comme au début de l'ère industrielle.

1. ASPEKTE DER DAUERHAFTIKEIT ALS PLANUNGSAUFGABE IM INDUSTRIEBAU

Auch bei Industriebauten traten in den letzten zwanzig Jahren zunehmend Bauschäden auf, die zu großem Kostenaufwand bei ihrer Beseitigung und bei der Sanierung führten. Gelegentlich erwies es sich als zweckmäßig, Teile der geschädigten Bausubstanz durch neue, andersartige Konstruktionen zu ersetzen. Dabei verwendete man anstelle der Materialien, die sich als nicht beständig erwiesen hatten, altbewährte, wie sie seit jeher im traditionellen Bauen verwendet worden waren.

Die Ursache solcher Schäden lag in einigen Fällen auch in den besonderen betrieblichen Zustandsbedingungen. Industriebauten dienen regelmäßig als Gebäudehülle für Produktion oder Lagerung. Dabei entstehen manchmal Emissionen, die bei gewöhnlicher Gebäudenutzung nicht auftreten. So können Gase und Dämpfe in Verbindung mit Schwitzwasser oder Luft den Baumaterialien durch chemische Reaktionen Schaden zufügen. Es gilt deshalb, die beim Betrieb eines Industriebaus zu erwartenden Verhältnisse vorher zu klären. Erwartete schädliche Einflüsse auf die Baukonstruktion sind durch geeignete Materialien und Konstruktionen zu vermeiden.

Dieser Beitrag behandelt als Teilproblem einen Aspekt der Dauerhaftigkeit von Dächern in der Alkohol-Getränkebranche.

2. PROJEKTSTUDIE UND GENERALBEBAUUNGSPLAN

Ein traditionsreiches deutsches Unternehmen der Alkohol-Getränkebranche ließ 1978/79 durch mich eine Projektstudie erstellen, aus der ein Generalbebauungsplan entstand. Das auf denkbare Bedürfnisse der Zukunft ausgerichtete Bauvolumen umfaßt ca. 300.000 cbm auf zwei verschiedenen Betriebsgeländen, von denen bisher etwa 75.000 cbm realisiert wurden.

Zur Projektstudie gehörten auch die Aspekte der Dauerhaftigkeit von Dachkonstruktionen unter den besonderen, branchentypischen Verhältnissen, da man bei bestehenden Bauten unterschiedlichen Alters sehr verschiedene Erfahrungen mit der technischen Beständigkeit von Dachkonstruktionen und Dachhaut gesammelt hatte. Die Einflüsse des bei der Destillatlagerung in Holzfässern verdunstenden Alkohols wirkten sich sehr unterschiedlich auf die Bausubstanz aus.

3. DÄCHER ALTER GEBÄUDE

3.1 Konstruktion

Traditionell wurden die geneigten Dächer über den Produktions- und Lagergebäuden der Alkohol-Getränkebranche als Holzkonstruktionen, teilweise mit weit gespannten Bindern ausgeführt. Auf einer Holzschalung befand sich eine Papplage und darüber eine Schieferdeckung, oder man legte gebrannte Tondachziegel auf einer Lattung aus. Derartige Dachkonstruktionen findet man ebenso im deutschen Rheingau, wie in Cognac, in der französischen Charente. Die wesentliche Ausprägung derartiger Dächer besteht in der schuppenden Dachdeckung, welche das Niederschlagswasser zuverlässig ableitet und zugleich dem emitierten Alkohol Gelegenheit gibt, durch unzählige Ritzen ins Freie zu entweichen.

3.2 Allgemeine Beurteilung der technischen Lebensdauer

Manche der bestehenden, in traditioneller Manier errichteten Lager und Produktionsgebäude sind über hundert Jahre alt. Die hölzerne Dachkonstruktion hat diese Zeit ohne Schaden überdauert. Die Dachhaut, d.h. die schuppende Deckung aus Schiefer oder Ziegeln, wurde durchschnittlich einmal erneuert, sonst nur bei kleineren Schäden repariert. Die erreichte technische Lebensdauer betrug bei Schieferdeckungen etwa achtzig Jahre, bei frostbeständigen, gebrannten Tonziegeln etwa neunzig bis einhundert Jahre und bei leichteren, französischen Dachziegeln etwa sechzig Jahre.

Da man jahrhundertealte, hölzerne Dachkonstruktionen kennt, ist davon auszugehen, daß diese bestehenden Dächer noch viele Jahrzehnte unbeschadet überdauern werden.

4. DÄCHER NEUER GEBÄUDE

4.1 Konstruktion

Die hier betrachteten Gebäude, welche nach 1960 entstanden sind, bestehen aus einer Stahlbeton-Fertigteilkonstruktion mit flachen oder flach geneigten Dächern. Auf den Bindern sind mit oder ohne Pfetten Stahltrapezbleche unterschiedlicher Spannweite in der Dachneigung oder parallel zu First und Traufe verlegt. Darüber befindet sich ein kontruktionsüblicher Walmdachaufbau, bestehend aus Dampfsperre, Wärmedämmung und oberer Haut, die aus geklebter Pappe oder aus aufgelegter Kunststoffbahn besteht.

4.2 Schadensbild und Ursachen

Bei diesen neueren Gebäuden entstanden schon wenige Jahre nach ihrer Fertigstellung, z.T. bereits nach drei Jahren des Gebrauchs, Schäden an der Dachhaut und in deren Folge auch an der Konstruktion. Stellenweise rosteten die Trapezbleche von den verborgenen Hohlräumen her durch und der Beton der Fertigteilkonstruktion wurde in den Randbereichen des Daches an der Oberfläche etwa 10 - 15 mm tief zerstört.

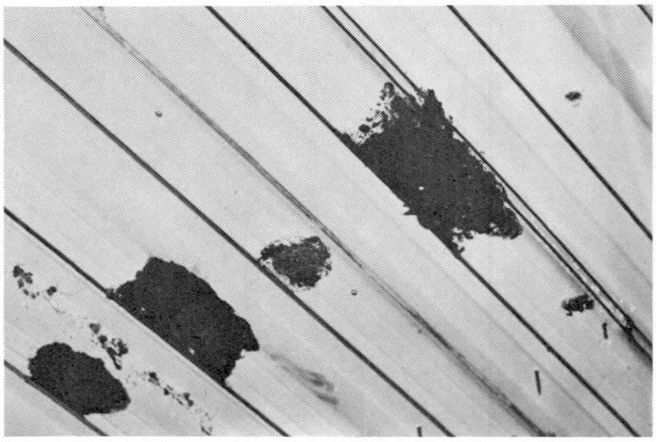


Abb. 1
Von den Hohlräumen her korrodierte Stahltrapezbleche des Daches einer Destillatlagerhalle. Essigsäure bildet die Ursache dieses Schadens.

Bei der Suche nach der Schadensursache stellte man fest, daß unterhalb des Daches der stechende Geruch von Essigsäure auftrat. Essigsäure, $CH_3.COOH$, bildet sich in den Hallen durch bakterielle Oxydation von verdünntem Alkohol an der Luft. Die besten Bedingungen liegen bei 5%- bis 10%igem Alkohol und 25° bis 35° C Lufttemperatur vor. Ebenso notwendig ist eine große Oberfläche und eine ständige Zufuhr von Luftsauerstoff. Dann entsteht Essigsäure:
$C_2H_5OH + O_2 \rightarrow CH_3COOH + H_2O$.

Bei $16{,}5^\circ$ C erstarrt Essigsäure zu eisähnlichen Kristallen, sogenannter Eisessig. Diese Kristalle sind in Wasser, Alkohol, Äther und Glyzerin in jedem Verhältnis löslich. Die Eisessigkristalle wurden ebenfalls in den Schadenbereichen der Halle angetroffen. Sie sind orange bis tiefrot gefärbt, was von der Eisenkorrosion der Armierung herrührt. Bei eingehenden Materialuntersuchungen fand man die Salze der Essigsäure, Azetate, vielfältig in den geschädigten Bereichen des Betons und an den korrodierten Metallteilen.

Alkohol ist stark hygroskopisch und mischt sich in jedem Verhältnis unter Volumenverminderung und gleichzeitiger Wärmeentwicklung. Es wird sich also überall dort verdünnter Alkohol als Ausgangsbasis für die Essigsäure bilden, wo Wasser in der Halle auftritt, z.B. als Folge von Wasserdampfkondensation (Schwitzwasser).

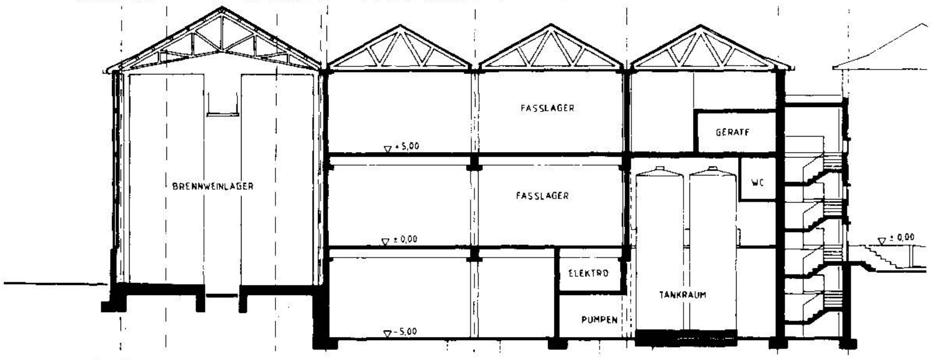
Die Sanierung der geschädigten Dächer erfolgt Zug um Zug so, daß die Stahltrapezbleche durch eine Holzkonstruktion mit Schalung ersetzt werden. Der Nachteil der insgesamt geschlossenen Warmdachfläche, welche das Entweichen des verdunstenden Alkohols in das Freie verhindert, ist dadurch jedoch nicht beseitigt.

5. NEUBAUTEN AB 1980

5.1 Planungskriterien

Für die Neuplanungen erfolgten Untersuchungen über die jeweils zweckmäßige Dachausführung unter Berücksichtigung der in den Produktions- und Lagerhallen herrschenden, besonderen Verhältnisse bezüglich Temperatur, Luftfeuchtigkeit und Alkohol-Konzentration, sowie des dadurch evtl. bewirkten Chemismus. Die bauliche Lösung erfolgte nach den optimalen, technisch-konstruktiven Möglichkeiten, der günstigsten Herstellungskosten, der Feuerwiderstandsklasse, der Wartungsfreundlichkeit der Bauteile und der absehbaren technischen Lebensdauer. Die inzwischen ausgeführten Objekte haben grundsätzlich geneigte Dächer, und sie sind mit einer schuppenden, wasserableitenden Deckung versehen, die jedoch nicht dampfdicht ist.

5.2 Objekt 1: Brennwein- und Destillatlager



Das Brennweinlager besteht aus zwölf zylindrischen Tanks (h=11,2 m, Ø=3,8 m). Das darüber befindliche, geneigte Dach wird von Stahlbindern getragen, wo sich zwischen diesen und oberhalb der Tanks die erforderlichen Arbeitsräume für Wartung und Betrieb ergeben. Die Dachkonstruktion über dem Destillatlager besteht aus hölzernen Nagelbindern mit einer Holzschalung, welche eine Papplage und die Schieferdeckung trägt. Weil an der Unterseite der Nagelbinder eine dem geforderten Feuerwiderstand entsprechende Gipskarton-Bekleidung angebracht wurde, entsteht ein wärmedämmender Dachhohlraum oberhalb der Lagerräume.

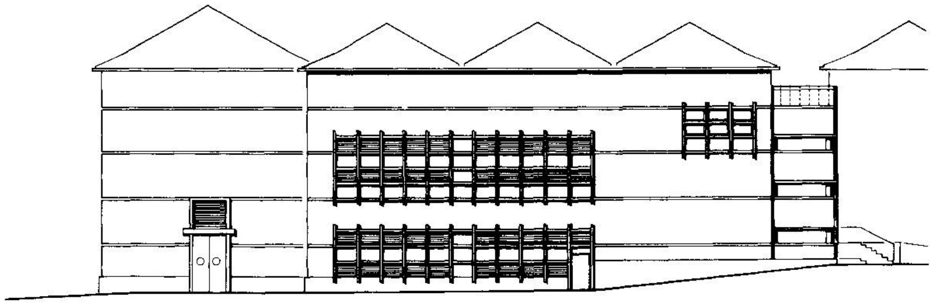
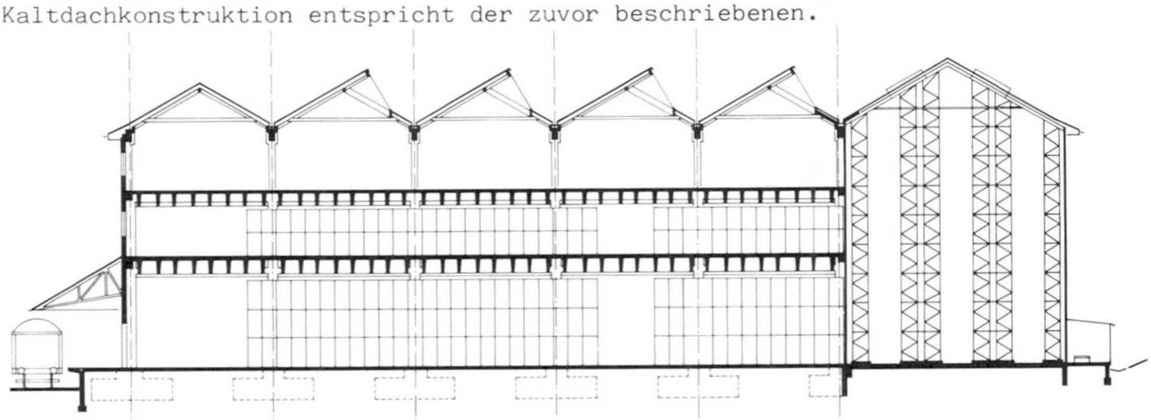


Abb. 2 Ein Industriebau mit ziegelgedeckten Walmdächern.

5.3 Objekt 2: Versandlager, Regallager

Das Versandlager besitzt oberhalb des aus Stahlbeton-Fertigteilen gefügten Baukörpers eine Konstruktion aus Holzleimprofilen und Bauholz. Das als Kaltdachkonstruktion ausgebildete Dach trägt die Wärmedämmung zwischen den Pfetten mit einem raumseitigen Feuerschutz aus Gipsplatten. Die Deckung besteht aus Dachziegeln auf einer Lattung. Neben dem Effekt des langlebigen Daches ergab sich auch eine Vergrößerung des inneren Raumvolumens, die, bezogen auf den ganzen nutzbaren Lagerraum des Gebäudes, 19% beträgt.

Oberhalb eines als Betriebsvorrichtung errichteten Regallagers befindet sich ein geneigtes Dach, welches direkt von den Regalstielen getragen wird. Die Kaltdachkonstruktion entspricht der zuvor beschriebenen.



5.4 Geplante technische Lebensdauer

Im Hinblick auf den zukünftigen Bauunterhaltungsaufwand wurde die technische Lebensdauer der Dachkonstruktion auf lange Zeiträume geplant. Unter der Voraussetzung, daß mechanische Beschädigungen und schädliche Einflüsse aus heute unbekannten Gründen nicht auftreten, kann mit dieser technischen Lebensdauer gerechnet werden:

- Schieferdeckung oder Ziegeldeckung	60 - 80 Jahre
- Gesamte Holzkonstruktion	100 Jahre
- Wärmedämmung und Brandschutzbekleidung	60 Jahre

5.5 Reparatur und Wartung

Im Falle mechanischer Beschädigungen an der Raumseite oder an der Außenseite der Dachhaut, lassen sich diese mit einfachen Mitteln vornehmen, z.B. Ersatz einzelner Schieferplatten oder einzelner Dachziegel. Beschädigte Brandschutzplatten auf der Rauminnenseite lassen sich ebenfalls leicht austauschen und neu anstreichen. Insgesamt ist die Konstruktion "fehlerfreundlich", d.h. auftretende Mängel der äußeren oder inneren Haut sind ohne großen Aufwand auf relativ einfache Art behebbar.

Die Dächer werden regelmäßig gewartet, d.h. sie werden ein- bis zweimal jährlich von Dachdeckern begangen, welche dabei zugleich kleinere Beschädigungen beheben. Sie tauschen beschädigte Ziegel aus, bessern die Vermörtelung der Firstziegel nach und beheben evtl. Frost- oder Sturmschäden.

Abb. 3
Geneigte Dächer vergrößern das nutzbare Raumvolumen.

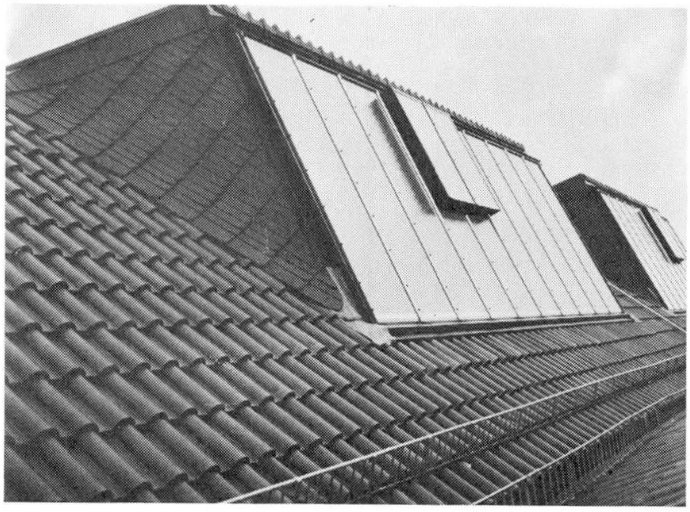


Abb. 4
Ziegelgedecktes Dach mit Shedlichtaufsätzen.

6. PLANBARKEIT DER DAUERHAFTIGKEIT

Die Dauerhaftigkeit, und damit die technische Lebensdauer von Bauteilen und Bauten ist planbar. Durch sorgfältiges Studium der in Industriebauten auftretenden betrieblichen Zustände und der Emissionen, lassen sich die jeweils optimalen Bauweisen und die geeigneten Materialien auswählen. Diese Kriterien treten zu denen der Gestaltung, der Tragwerksplanung und der allgemeinen, bauphysikalischen Durchbildung eines Bauwerkes hinzu.

Bewertung von Betonschäden unter wirtschaftlichen Aspekten

Evaluation of Concrete Damage under Economical Aspects

Évaluation de dommages du béton en tenant compte des aspects économiques

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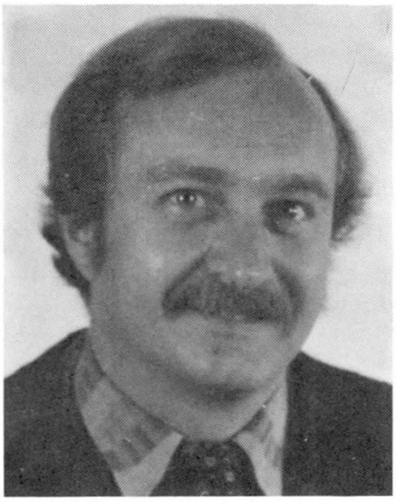

Wolfgang Lenkeit, geboren 1950, promovierte als Bauingenieur an der Technischen Universität Berlin. Nach zweijähriger Ingenieurbürotätigkeit wurde er wissenschaftlicher Mitarbeiter an der TU Berlin, wo er in der Forschung und als Gutachter tätig war. Seit 1984 ist er Mitinhaber des Betontechlogischen Institutes. Er befasst sich mit Gutachten, Diagnose, Ausschreibung und Bauleitung bei Bauschäden und Instandsetzungsmassnahmen.

ZUSAMMENFASSUNG

Eine vorbeugende Sanierung von Betonaussenbauteilen ist nur dann erforderlich, wenn karbonatisierungsbedingte Korrosionsschäden während der Nutzungsdauer des Bauwerkes zu erwarten sind. In einem solchen Fall ist es immer wirtschaftlicher, eine vorbeugende Instandsetzung durchzuführen, als die Korrosionsschäden abzuwarten, um sie dann beseitigen zu müssen. Diese vorbeugende Instandsetzung sollte in wirtschaftlicher Hinsicht soweit wie technisch möglich hinausgezögert werden.

SUMMARY

A preventive coating of outdoor concrete elements is only needed when damage due to carbonation will occur in the service time of the building. In such a case it is always more economical to apply a preventive coating than to wait till the corrosion occurs and repair the damage afterwards. The preventive coating is most economical when postponed as much as is technically possible.

RÉSUMÉ

Une protection préventive d'éléments extérieurs en béton n'est nécessaire que si des dommages dûs à la carbonatation se produisent pendant la durée de service du bâtiment. Dans ce cas, il est toujours plus économique d'appliquer préventivement une couche protectrice que de réparer les dommages. La mesure protectrice différée aussi longtemps que techniquement possible sera la plus économique.

1. ZWECK UND ZIEL

Aufgrund der in den letzten Jahren vermehrt aufgetretenen Korrosionsschäden in Betonaußenwänden ist es erforderlich geworden, die zur Schadensbehebung und eventuell zur Schadensvorbeugung erforderlichen finanziellen Mittel dem vorhandenen Etat für Sanierungen gegenüberzustellen, um anschließend Prioritäten setzen zu können. Bei der Ausarbeitung der Sanierungspläne ist die Kenntnis erforderlich, inwieweit z. B. eine vorbeugende Maßnahme wirtschaftlicher ist als eine spätere Sanierung - bei der die Kosten überproportional steigen werden.

Ziel ist es, karbonatisierungsbedingte Schäden in Schadensklassen einzuteilen und darauf aufbauend Grundlagen zu erarbeiten, die die Ermittlung der wirtschaftlichsten Sanierungsmaßnahmen entsprechend der jeweiligen Schadensklasse zu ermöglichen [1].

Bei den folgenden Ausführungen wird vorausgesetzt, daß sowohl die Ursachen von karbonatisierungsbedingten Korrosionsschäden als auch deren Instandsetzung dem Leser bekannt sind und deswegen nicht erläutert werden.

2. EINTEILUNG IN SCHADENSSTUFEN

2.1 Übersicht

In Abhängigkeit von dem karbonatisierungsbedingten Beschädigungsgrad und von den erforderlichen Sanierungmaßnahmen können Stahlbeton-Außenbauteile in die vier folgenden Schadensstufen eingeteilt werden:

- Schadensstufe I: keine Schäden, Sanierung nicht erforderlich
- Schadensstufe II: keine Schäden, vorbeugende Sanierung erforderlich
- Schadensstufe III: Korrosionsschäden vorhanden, Tragfähigkeit nicht gefährdet
- Schadensstufe IV: starke Korrosionsschäden vorhanden, Tragfähigkeit gefährdet.

Die Einteilung in diese Schadensstufen erfolgt unter Zugrundelegung einer repräsentativen Schadensdiagnose. Die Auswertung der Meßergebnisse soll ingenieurmäßig erfolgen, unter Berücksichtigung der vorhandenen Streuung der Meßergebnisse, Gefährdungsgrad und Orientierung des Bauteils usw.

Durch die Einteilung in Schadensstufen wird eine schnelle Beurteilung der Konstruktion hinsichtlich der vorhandenen karbonatisierten Korrosionsschäden im Hinblick auf die erforderlichen Sanierungsmaßnahmen ermöglicht. Darüber hinaus kann beurteilt werden, ob und wann eine vorbeugende Sanierung durchzuführen ist. Der Entscheidungsprozeß ist in Bild 1 dargestellt.

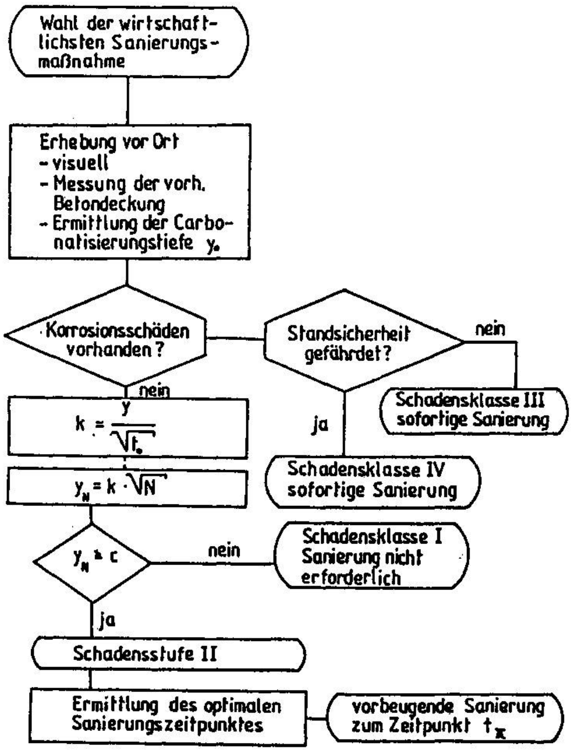

Bild 1 Schematische Darstellung des Entscheidungsvorganges

Die im folgenden verwendete Berechnungsmethode zur Klassifizierung eines Bauteiles in die jeweiligen Schadensstufen beruht auf empirischen Untersuchungen unter Verwendung der in [2] gegebenen Gleichung für den Karbonatisierungsverlauf in Abhängigkeit von der Zeit (s. Gl. (1)). Auch wenn diese Gleichung hinsichtlich ihrer Genauigkeit umstritten ist, ist sie für die hier beabsichtigten Zwecke - nämlich Entscheidungsgrundlage für die Wahl der richtigen Sanierungsmaßnahme - ausreichend.

Die im folgenden getroffenen Aussagen für die Schadensstufen I und II sollten ohnehin in festen Abständen (z. B. alle fünf Jahre) überprüft werden, da der Gefährungsgrad eines Bauteiles sich im Laufe der Zeit verändern kann (z. B. aufgrund veränderter Verkehrsführung, Nutzungsänderungen usw.).

2.2 Definition der einzelnen Schadensstufen

2.2.1 Schadensstufe I: keine Schäden, Sanierung nicht erforderlich

In dieser Schadensstufe werden diejenigen Stahlbeton-Außenbauteile eingestuft, in denen die Karbonatisierungstiefe während der gesamten Lebensdauer des Bauwerkes immer kleiner als die Betonüberdeckung sein wird. Dies ist in der Regel der Fall bei sehr guten Betonen (B 35 und höher) und bei Vorhandensein einer ausreichenden Betonüberdeckung der Stahlbewehrung.

Die Ermittlung der vorhandenen Betonüberdeckung c und der Karbonatisierungstiefe y_0 ist in Abschnitt 2 beschrieben. Der weitere Fortgang der Karbonatisierung kann nach [2] wie folgt berechnet werden:

$$y = k \cdot \sqrt{N} \text{ [mm]} \quad (1)$$

Es bedeuten:

k Karbonatisierungskonstante $\left[\frac{mm}{a^{0.5}}\right]$

$$k = \frac{y_0}{\sqrt{t_0}} \quad (2)$$

N Lebensdauer des Bauwerkes [a]

t_0 Alter des Bauwerkes zum Erhebungszeitpunkt [a]

y Prognostizierte Karbonatisierungstiefe zum Zeitpunkt N [mm]

y_0 Karbonatisierungstiefe zum Erhebungszeitpunkt [mm]

Wenn die prognostizierte Karbonatisierungstiefe kleiner als die vorhandene Betonüberdeckung ist, so können karbonatisierungsbedingte Korrosionsschäden während der Lebensdauer des Bauwerkes ausgeschlossen werden. Eine vorbeugende Sanierung in der Form eines karbonatisierungsbremsenden Anstriches wäre in diesem Fall nicht erforderlich.

Wenn jedoch die prognostizierte Karbonatisierungstiefe größer als die vorhandene Betonüberdeckung ist, so muß mit späteren Korrosionsschäden gerechnet werden. - In diesem Fall ist das Bauteil der Schadensstufe II zuzuordnen.

2.2.2 Schadensstufe II: keine Schäden, vorbeugende Sanierung erforderlich

In dieser Schadensstufe werden diejenigen Stahlbeton-Außenbauteile klassifiziert, in denen die Karbonatisierungstiefe zwar zum Entscheidungszeitpunkt kleiner als die Betonüberdeckung ist, aber während der Lebensdauer des Bauwerkes karbonatisierungsbedingte Korrosionsschäden zu erwarten sind. Das bedeutet, daß während der erwarteten Lebensdauer des Bauwerkes die Karbonatisierungstiefe (s. Gl. (1)) größer als die vorhandene Betonüberdeckung sein wird, und daß dadurch aufgrund des dann nicht mehr vorhandenen Korrosionsschutzes voraussichtlich Korrosionsschäden auftreten werden.

Die in [1] dargestellten Untersuchungen haben ergeben, daß eine vorbeugende Sanierung immer billiger ist als das Auftreten von Korrosionsschäden abzuwarten, um sie nachträglich zu sanieren. Hierbei wurden unterschiedliche Sanierungskosten, unterschiedliche Zinsen sowie Baupreissteigerungsraten und die Tatsache, daß karbonatisierungsbremsende Anstriche kreiden - wie alle anderen Anstriche auch - und deswegen ca. alle 10 Jahre zu erneuern sind, berücksichtigt (siehe Bild 2).

Aufgrund der regelmäßigen Erneuerungsbedürftigkeit des Anstriches würde eine zu frühe Sanierung mehr Anstriche bedingen; eine zu späte vorbeugende Sanierung kann u. U. die karbonatisierungsbedingten Korrosionsschäden nicht mehr abwenden. Das bedeutet, daß einerseits das Hinauszögern der vorbeugenden Sanierung die Kosten verringert, aber andererseits, wenn die vorbeugende Sanierung nicht rechtzeitig eingeleitet wird, spätere und auch teurere Schäden nicht mehr vom Bauwerk abgewendet werden können.

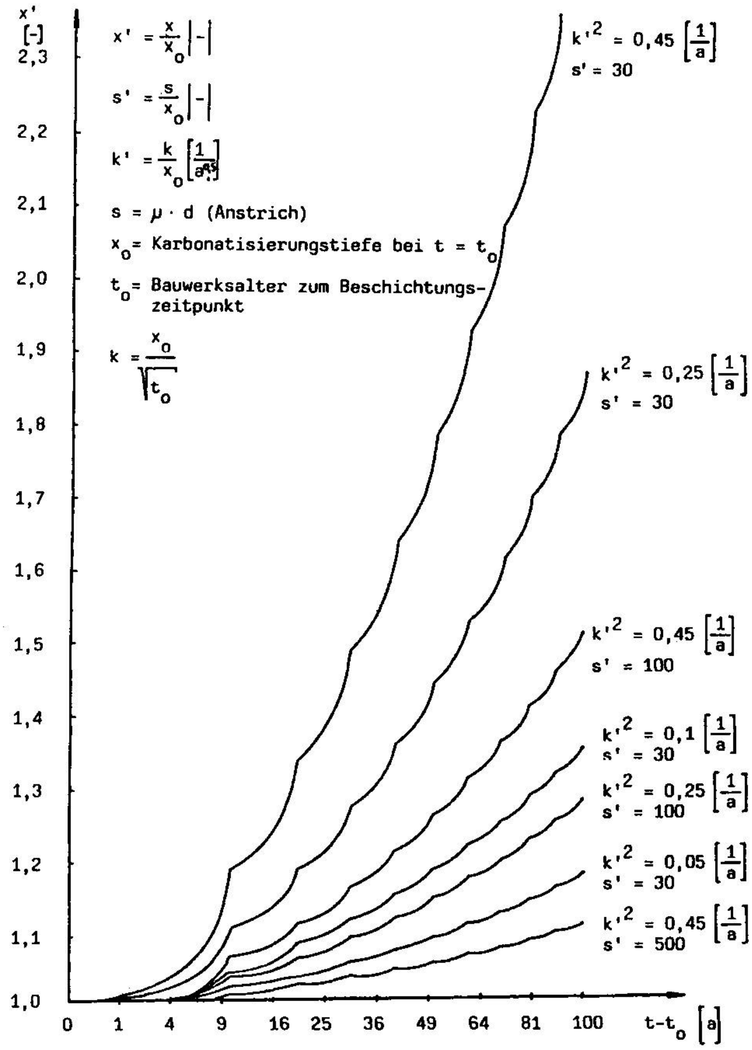

Bild 2 Karbonatisierung in Abhängigkeit von der Zeit unter Berücksichtigung der Kreidung des Anstriches. Auszug aus [2]

Der späteste und somit auch optimale Zeitpunkt für die vorbeugende Sanierung erfolgt nach [1] zu:

$$t = \left(\frac{c}{k} - \frac{k}{\alpha}\right)^2 \quad [a] \qquad (3)$$

Es bedeuten:

t	optimaler Sanierungszeitpunkt	[a]
c	minimal vorhandene Betonüberdeckung	[mm]
$k = \frac{y_0}{\sqrt{t_0}}$		$\left[\frac{mm}{a^{0.5}}\right]$
y_0	Karbonatisierungstiefe zum Erhebungszeitpunkt	[mm]
t_0	Alter des Betons zum Erhebungszeitpunkt	[a]

α = 8,2 für N = 100 [a]

α = 10,3 für N = 80 [a]

α = 16,5 für N = 50 [a]

N = Lebensdauer des Bauwerks [a]

2.2.3 Schadensstufe III: Korrosionsschäden vorhanden, Tragfähigkeit nicht gefährdet

In dieser Schadensstufe werden diejenigen Stahlbeton-Außenbauteile eingestuft, in denen die Karbonatisierungstiefe bereichsweise größer als die Betonüberdekkung der Stahlbewehrung ist (y > c) und deswegen bereits korrosionsbedingte Absprengungen der Betonüberdeckung vorhanden sind. Die Korrosionsschäden dürfen hierbei jedoch nicht so weit fortgeschritten sein, daß eine Gefährdung der Standsicherheit gegeben ist.

Bauteile dieser Schadensstufe weisen korrosionsbedingte Abplatzungen der Betonüberdeckung auf. - Aufgrund dieser Betonabplatzungen ist es erforderlich, die Sanierungsmaßnahmen kurzfristig einzuleiten: Wenn Stahl ungeschützt im Abplatzungsbereich direkt der Atmosphäre ausgesetzt ist bzw. im bereits karbonatisierten Betonbereich liegt, kann er schnell korrodieren. Aufgrund der deswegen zunehmenden Korrosionsschäden wird der erforderliche Aufwand, um die Schäden zu beheben, in kürzester Zeit überproportional steigen (siehe Bild 3).

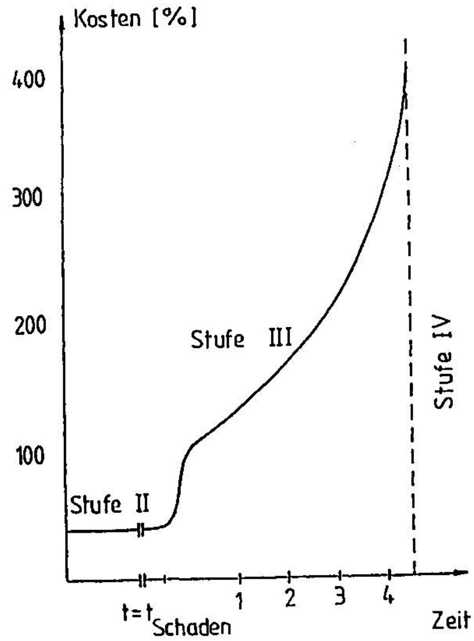

Bild 3 Entwicklung der Instandsetzungskosten für Beton in Abhängigkeit von der Zeit bei nicht erfolgter Instandsetzung

2.2.4 Schadensstufe IV: Korrosionsschäden vorhanden, Tragfähigkeit gefährdet

In dieser Schadensstufe werden diejenigen Stahlbeton-Außenbauteile klassifiziert, in denen die karbonatisierungsbedingte Korrosion der Stahlbewehrung so weit fortgeschritten ist, daß die Standsicherheit nicht mehr gewährleistet ist.

Bauteile dieser Schadensklasse sind aufgrund der nicht ausreichend vorhandenen Standsicherheit sofort zu sanieren. Entsprechend der statischen Bedeutung des Bauteiles und in Abhängigkeit vom Zerstörungsgrad der vorhandenen Stahlbewehrung sind gegebenenfalls sofortige Stützmaßnahmen erforderlich. Die Standsicherheit muß selbstverständlich auch während der Sanierung gewährleistet sein.

Diese Schadensstufe wird deswegen erreicht, weil Bauteile der Schadensstufe III nicht rechtzeitig saniert worden sind.

3. LITERATUR

[1] Friedmann, M.: Betonkorrosion im Hochbau; Schadensanalyse, Betonvoruntersuchung, Bewertung - Bauverlag, 1986.

[2] Klopfer, H.: Die Carbonatisierung von Sichtbeton und ihre Bekämpfung. Bautenschutz und Bausanierung, Heft 3, 1978.

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Structural Design of Liyutan Railway Bridge

Conception du pont ferroviaire de Liyutan

Projektierung der Liyutan – Eisenbahnbrücke

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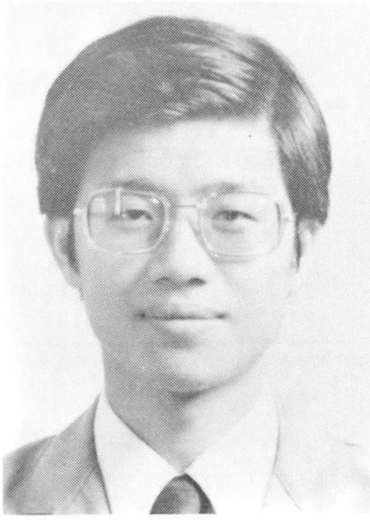

Mr. S. J. Lin, born in 1931, received his B. S. Degree in Civil Engineering from the National Cheng Kung University, Tainan, Taiwan. Since graduation he has devoted himself to the design and supervision of highway bridges and other structures for 32 years. He is now responsible for the design and supervision of the firm's multi-disciplined engineering services.

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Mr. J. L. Wang, born in 1953, received his M. S. Degree in Civil Engineering from the National Cheng Kung University, Tainan, Taiwan. Since graduation he has devoted himself to the design of bridges and buildings for 9 years.

SUMMARY

The Liyutan Railway Bridge, a continuous four-span, reinforced concrete bridge with an inverted Langer arch, will be the longest continuous rigid frame railway bridge in Taiwan after its completion. This paper describes such technical information as the member arrangement, structural analysis, seismic response and construction method for the bridge.

RÉSUMÉ

Le pont ferroviaire de Liyutan, un pont arc en béton armé continu sur quatre travées, sera le plus long pont continu sans appui glissant. Cet article décrit le système statique, le dimensionnement, le comportement au séisme et l'exécution de l'ouvrage.

ZUSAMMENFASSUNG

Die Liyutan-Eisenbahnbrücke, eine über vier Spannweiten durchlaufende Bogenbrücke aus Stahlbeton, wird bei ihrer Vollendung die längste festgelagerte Eisenbahnbrücke Taiwans sein. Der Beitrag beschreibt das Tragsystem, die Berechnung, das Erdbebenverhalten und den Bauvorgang.

1. INTRODUCTION

Liyutan Bridge, the main bridge in the extension project of the Taiwan Railway is located in the hilly region of central Taiwan. It crosses over a valley 740 meters in width and connects two tunnels one on each end. A design elevation with a value of about 40 meters across the width of the valley with a range of 400 meters will be used to design the bridge

In addition, the bridge is surrounded by beautiful, calm and peaceful mountain scenery. The construction of the arch bridge will attract more tourist to enjoy the scenic beauty of this region. At a distance of 4.5 kilometers from the bridge in this valley, a reservoir is being built. The region around the reservoir will also be developed into a recreational and tourist area.

During the planning stage, five types of bridge alternatives were proposed, as illustrated in Fig. 1. But after investigation and comparision with regard to structural appearance, durability, economic feasibility, construction method, etc. Type 1, the continuous, reinforced concrete, inverted Langer arch bridge, was chosen.

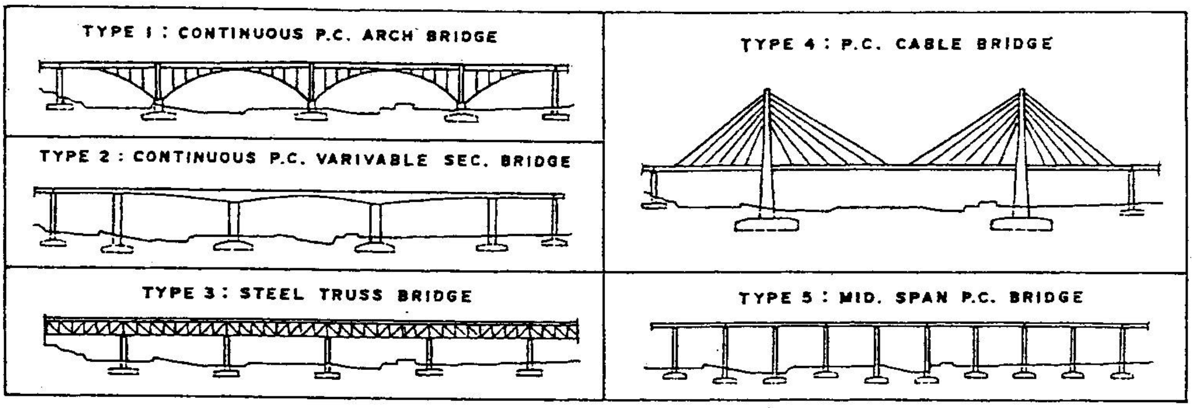

Fig. 1 Five bridge alternatives during the planning stage.

2. DESCRIPTION OF BRIDGE

The four-span bridge comprises two 134-meter central spans and two 70-meter side spans. Its general configuration is shown in Fig 2. Its box girder, supporting two railway tracks, will be supported by spandrel struts and arch ribs.

The box girder, besides resisting design moment after completion, is also designed with some prestressed tendons to satisfy the requirements in the segmental construction method. To reduce stress from creep and shrinkage and to increase the ductility of the bridge, the ratio of prestressed tendons will not be large. In addition, top slabs are prestressed transversely to reduce the self-dead load. To resist the shear and increase the durability during earthquake, the web thickness near by the pier column will be made larger than that in the center of the bridge.

Generally, a circular arc has been adopted for the arch rib as its fundamental configuration. However, curvature near by the pier column was modified following detailed analysis and comparison to reduce the moment and self-dead load in the arch rib and to exert the mechanical advantage of the arch bridge as a pure compression member.

In consideration of stability, spandrel struts are designed not only to satisfy all mechanic requirements, but also to fulfil the consideration of slenderness effect.

For increased durability, the outer bridge areas exposed to weathering will be painted with a layer of protective painting. In addition, a concrete strength higher than that of ordinary bridges will be adopted.

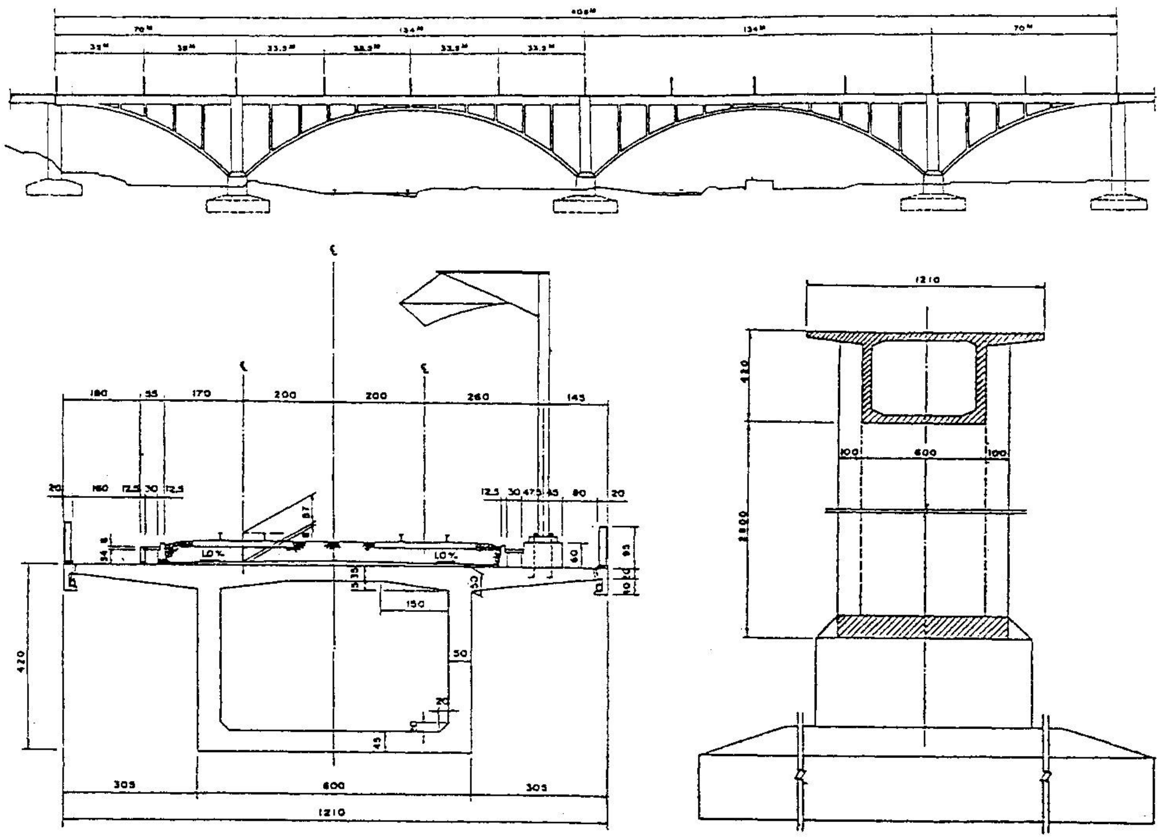

Fig. 2 General configuration of the bridge (Type 1).

3. STRUCTURAL ANALYSIS

Main design criteria and design data are illustrated in Table 1. Two-dimensional linear elastic analysis was employed to evaluate the sectional forces due to vertical loading, earthquake force, wind loading, creep, shrinkage and temperature changes along the bridge. Besides stress analysis of the bridge subjected to loading after completion, sectional forces from dead load during the process of construction will be calculated. Futhermore, the load carrying capacity of bridge will also be checked.

At present, electrified locomotives are used throughout the Taiwan Railway system. To avoid separation of electric cables from above the train, the railroad bridge must be designed with sufficient stiffness to limit the deflection and sidesway of the bridge. In addition, vertical deflection from vertical loading and horizontal sidesway from horizontal earthquake force as shown in Fig 3 and Fig 4 are limited to 0.84 cm and 9.6 cm, respectively.

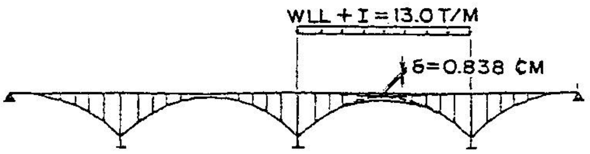

Fig. 3 Vertical deflection due to vertical loading (dead load and live load).

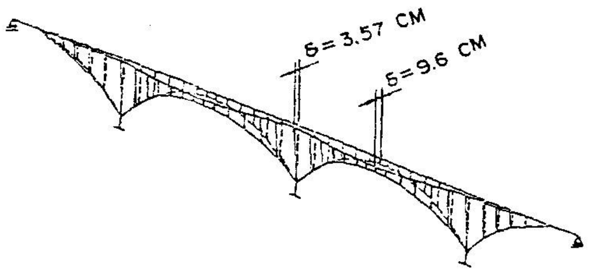

Fig. 4 Horizontal deflection due to horizontal earthquake force.

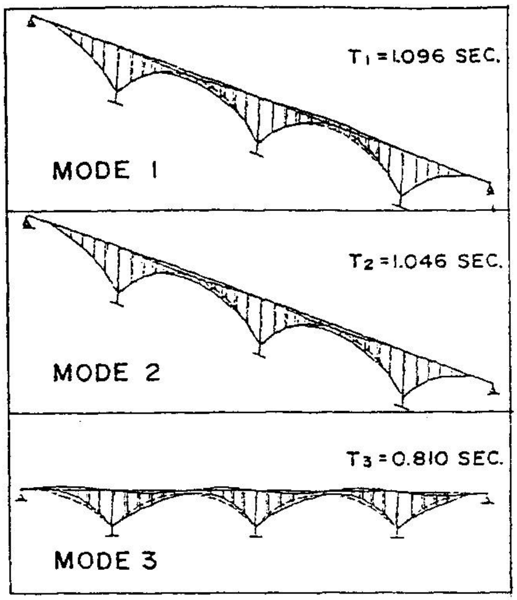

Fig. 5 The first three mode shapes and its natural period in the dynamic analysis.

Design Load	: KS 18	Concrete Strength	: $f'c$ = 350 Kg/cm^2
Seismic Coefficient	: Kh = 0.25	Reinforcement	: fy = 2800 Kg/cm^2
	Kv = 0.10	Prestressed tendon	: $f's$ = 19000 Kg/cm^2
Temperature Change	: ± 15°c	Creep coefficient	: Ø = 2.6
Design Wind Speed	: 65 m/sec	Shrinkage strain	: ε_s = 2.0x10

Table 1 Design data.

4. SEISMIC ANALYSIS

According to seismic records, the proposed site is located in a seismically active region. For example, one destructive earthquake with a magnitude of M 7.2 took place within a distance of 20 kilometers from the bridge site in 1935. In order to study the dynamic behavior of the bridge, seismic analysis was performed. The first three mode shapes and their natural periods are shown in Fig 5. From dynamic analysis, a design seismic force coefficient of 0.25 was decided for the earthquake design.

5. DESIGN OF FOUNDATION

According to the results of geologic investigations, a gravel layer with a thickness of 12 meters underground was found near Liyutan Bridge. Under the gravel layer is a layer of sandstone. Hence, the bearing capacity of the soil in the site is good. Therefore, spread footing is to be used as the foundation for the bridge.

6. CONSTRUCTION METHOD

The Cantilever Segmental Construction method with a moveable wagon mounted on the main girder will be used during erection. Through this method, the main girder, the arch rib, and the vertical struts may be erected without use of support. The erection sequence is shown in Fig 6.

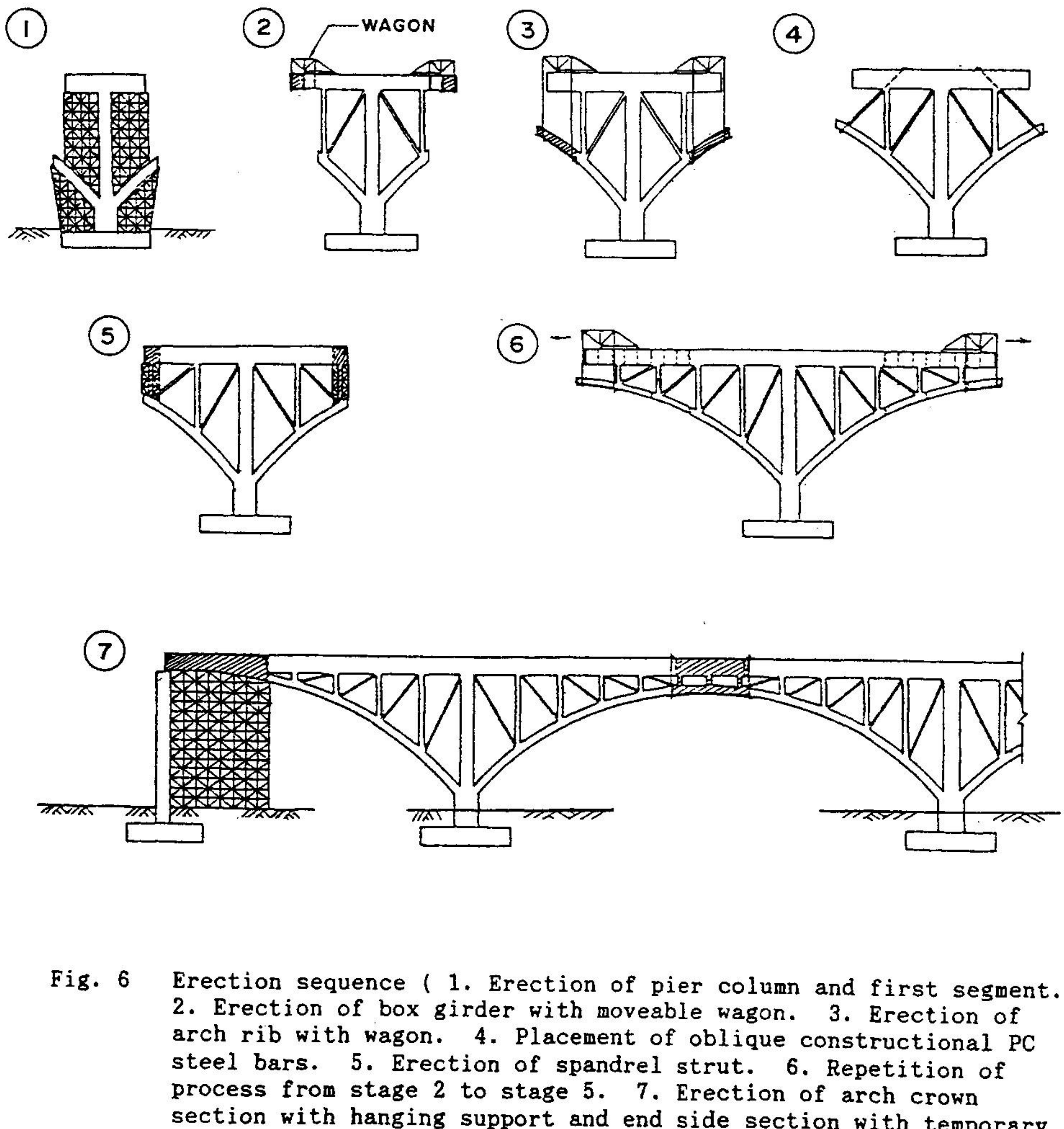

Fig. 6 Erection sequence (1. Erection of pier column and first segment. 2. Erection of box girder with moveable wagon. 3. Erection of arch rib with wagon. 4. Placement of oblique constructional PC steel bars. 5. Erection of spandrel strut. 6. Repetition of process from stage 2 to stage 5. 7. Erection of arch crown section with hanging support and end side section with temporary support.).

7. CONCLUSION

Data collection and planning of this project started in Oct. 1987. The final alternative was selected and designed in 1988. Construction is planned to begin in May 1989 and all works are to be finished within 36 months. Fifteen months were used to complete the design work, from the time of planning to the time of finish. During the design process, all factors such as structural appearance, strength, durability, economic feasibility, construction method,etc. were considered thoroughly.

Data Bank System for the Maintenance of Highway Structures

Banque de données pour la gestion des ouvrages d'art

Datenbanksystem für die Erhaltung von Kunstbauten

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SUMMARY

The data bank management system permits the computerized registration of structures including checking and repair work, provides an effective means of control and monitoring by the responsible authorities and simplifies the work of checking the structures. It was developed as a joint project commissioned by the Swiss Federal Highways Department together with Canton Schwyz as pilot canton.

RÉSUMÉ

La banque de données permet de gérer par l'informatique les données relatives aux ouvrages d'art y compris les contrôles et les assainissements. Elle est un outil efficace de contrôle et de surveillance pour les autorités responsables et simplifie les inspections des ouvrages d'art. Le logiciel a été développé par un groupe de travail sous la direction de l'office suisse des routes en collaboration avec le canton de Schwyz pour le projet pilote.

ZUSAMMENFASSUNG

Das Datenbank-System ermöglicht die Erfassung von Kunstbauten mittels EDV inklusive Kontrollen und Sanierungsmassnahmen, unterstützt als wirkungsvolles Führungsmittel die Kontroll- und Ueberwachungsaufgabe der zuständigen Aufsichtsbehörden und erleichtert die Durchführung der Bauwerkskontrollen. Es wurde durch eine Arbeitsgemeinschaft unter Aufsicht des Schweizerischen Bundesamtes für Strassenbau zusammen mit dem Kanton Schwyz als Pilotkanton, entwickelt.

PROBLEM DESCRIPTION

A large part of the present Swiss highways system was constructed during the last 40 years. The maintenance of these highways presents the owners and those responsible for their upkeep with new problems particularly in connection with the numerous structures involved, there being altogether about 10,000 structures of which about 3,000 are bridges on the National Highways network alone. With the aid of a new data bank management system this maintenance work should in future be both simplified and unified.

The data bank for the maintenance of structures (structures data bank) serves the cantons as a practical tool and guide in the work of structural maintenance and helps the Federal Highways Department in its nationwide supervisory responsibilities. In order to include the specific wishes and needs of the cantons, Canton Schwyz was brought in and heavily involved in the development of the structures data bank.

Data exchange between the cantons and the Swiss Federation must be guaranteed despite different computer euqipment and greatly varying degrees of hardware installation. At the moment this data exchange will be carried out annually using diskettes as a storage medium. A proper data network system requires, in addition to the inevitable standardisation of data contents and data structures, also compatibility in the data administration systems.

CONCEPT

The data bank management system must be capable of running on mainframes, mini--computers and IBM compatible PC's both as single user and in a network configuration. For personal computers the following are the minimum requirements:

- main memory	1.5 MB
- fixed disk storage	40 MB
- floppy drives	1.2 MB

For the problem at hand only a data bank software of a relational type is feasible. The data is organized in the form of relations (a mathematical concept). In general, this can be visualised as two-dimensional tables. The special feature of the relational model is that the user data and the relations are strictly separated. In this way the system may be extended and the disadvantage of hierarchical models in which the access path is given in data is thus overcome. So also in the planning stage one can still accomodate unforeseen questions. The capability of running on various different computer systems and especially on PC's was the deciding factor in the choice of ORACLE as the data bank software.

Special demands are placed on the user comfort. In this respect ORACLE did not satisfy all the requirements. As a result ORACLE was enhanced to have a user guide and a user interface developed specifically for the structures data bank, which included screen formats in various colours and windowing.

Additional user-friendliness was achieved by the implementation of the following important aids:

- automation of the data saving, access and function selection
- extendable catalogs with expert knowledge
- standard access paths and corresponding lists for quick access
- automatic output of structural element specific checklists
- structural component generators

To achieve good maintenance characteristics the application parameters and all text with the exception of the free texts are kept in external lists. Particular significance in this respect was given to accomodating several official languages as is necessary in a country like Switzerland by having text-independent data storage. In working with the structures data bank each user can choose between the German, French, and Italian languages.

EXTERNAL DATA SCHEME

The external data scheme, Figure 1, shows, from the standpoint of the user, the structuring and view of the data, which is subdivided into object data and structural component data. With the component classes given in Figure 1 and 2 similar or related structural components are combined. These element or component classes, however, do not contain user data but the catalog information at the component level. When possible the data processing is supported by the catalog information with expert knowledge stored in the system.

Both object and structural component data refer to three different areas, namely

- description
- checking
- repair work

The history of the structure can be reconstructed out of the classified data. This has the advantage that the relationship of checking to repair work is clearly seen and later evaluations of all kinds of structural components with regard to durability and maintenance expenditures can be undertaken. A new description of this structural component is only added to the system if the component has been replaced or substantial structural changes, i.e. changes relevant to the description, have been made. If the complete structure (object) has been replaced or correspondingly rebuilt or changed, then a new object has to be defined.

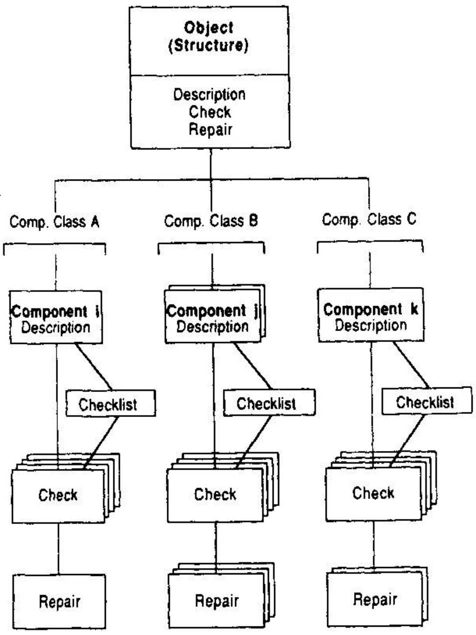

Fig. 1: External data scheme

Object data

The object data includes all data relevant to the whole structure. The description comprises at the object level, Figure 1, information concerning:

- reference systems
- basic data: object type, function
system, geometry
year of construction
participants
checking cycles
- documents: standards
construction documents

Each individual measure for checking and repair work ist anchored as follows:

- checking: type of checking, number
  year when checking carried out
  participants
  condition assessment

- repair work: repair number
  year of repair work
  participants

Apart from the time reference system (calendar year) the objects are linked with three further reference systems:

- hierarchical system (owner)
  country, canton
  object number

- operating system: those responsible for maintenance
  maintenance section

- axis system (highways etc.):
  for highways:
  highway key
  ref. point, kilometres
  position of axis
  restrictions on use

The object key is obtained from these reference systems by means of the hierarchical system. To each object or structure can be assigned a maintenance section with those responsible for maintenance as well as any axis, of which at least one has to be a highway axis with defined highway key (e.g. cantonal highway, national highway). The highway axis defines in this way the object position, with details of a reference point and kilometres. With the axis position the position is also given of the highway axis relative to the structure, and with the information on restrictions concerning use (geometry and loads) the data bank is also provided with the basic data on whether the structure is open to exceptional transport.

Thus far the following object types are implemented in the system:

- bridge
- gallery
- cut and cover tunnel
- culvert
- retaining structure
- protective structure

Structural component data

Each object or structure can be subdivided into a number of structural components, whereby each component is assigned to a particular (structural) element or component class, which enables similar or related structural components to be placed together.

Figure 2 shows the componet classes implemented thus far in the system. Here element classes connected with the bearing structure are distinguished from other element classes, which mainly involve structural components having high maintenance expenditure.

It would exceed the scope of this article to describe in detail all the element classes included in Figure 2 together with their corresponding structural components. Instead, it is more appropriate here to illustrate using the example of the structural component "i" of a bridge superstructure the main relationships between this structural component and its element class (Figure 3).

All user data is related basically to the structural component. The element class contains a selection of the stored catalog information with expert knowledge. In the data processing a corresponding catalog choice appears in the screen window. Depending on the case considered one or more details may be assigned to the structural component using keypress.

Component Class	Bridge	Other Structure
Foundations	□	–
Burried Structural Comp.	X	X
Abutments	X	–
Colums, Piers, Pylons	X	–
(Super) Structure	□	–
Bearing Structure	–	□
Bearings, Joints	X	X
Expansion Joints	X	X
Sealing	X	X
Surfacing (Pavement)	X	X
Safety Installations	X	X
Concrete-Surface Treatment	X	X
Carriageway Drainage	X	X
Installations	X	X

□ always X if existent

Fig. 2: Component classes

A furhter speciality of this system are the stored defects catalogs (Fig. 3), which are classified according to component classes and structural materials. After defining the structural materials the material-related defects are automatically selected, so that a component- and material-specific checklist is generated. It can be printed and it simplifies in this form the checking of the structural components on site. In addition, this checklist appears on the screen and aids the computer investigation of defects. For each structural component any combination of defects can be stored for arbitrarily chosen positions.

The evaluation of defects carried out for each position on a structural component is summerized to the level of object, in order to pass on relevant information to all those responsible.

The inclusion of the rapair measures is seen in Figure 3. If changes are made to a structural component, i.e. with replacement or reconstruction affecting the description, then this component has to be described once again.

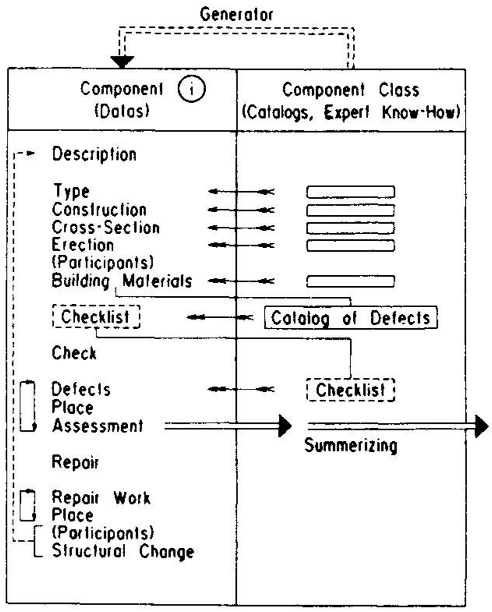

Fig. 3: Component of superstructure

CONCEPTIONAL DATA SCHEME

The conceptional data scheme differs from the external scheme in that it shows the structural relationships between data types, relations and catalog information. In designing the conceptional data scheme care was given to summarizing all user data in a few compact tables, in order to keep the question search time to a minimum. The development of the system architecture is based on the most up-to-date principles: user data, catalog information and relations in the data bank for the maintenance of highway structures are separate from each other.

FINAL REMARKS

The data bank management system described may be easily extended. Some extensions are already planned, whereas others are conceivable, such as a statistical evaluation of defects, assessment of structural types or repair measures, expert proposals for repair works and budget planning.

Data bank and expert systems will undoubtedly establish themselves in the future as additional instruments for the engineer. The meaningful and economic use of such systems, however, will be restricted to those who have a broad and fundamental technical knowledge. This and the ability to think in terms of structured and interrelated systems (networking) will be more important for the future user than special computer know-how.

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