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SESSION 4

a) Joints, Bearings and Other Details
b) Financial and Planning Considerations

a) Joints, appuis et autres équipements annexes
b) Aspects financiers et planification

a) Fugen, Auflager und andere konstruktive Elemente
b) Finanzielle Aspekte und Planung

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Design, Performance and Maintenance of Bearings and Joints

Conception, exécution et entretien d'appuis et de joints

Planung, Ausführung und Unterhaltung von Auflagern und Fugen

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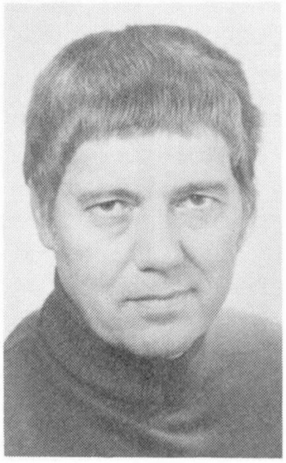


Waldemar Köster, born in 1923, studied civil engineering at the Technological University of Darmstadt, Fed. Rep. of Germany. He worked twelve years in the bridge design offices of major construction firms at Cologne. Since 1966 Waldemar Koster is a consultant to the expansion joint and bearing industry and holds numerous world patents on their design.

SUMMARY
Bearings and joints are subject to movement processes, for which they are normally mechanically designed. In this way, modern bridges have become a mechanical feature. Problems of kinematic and dynamic nature will be discussed.

RESUME
Les appuis et joints de dilatation sont soumis à des mouvements en vue desquels ils ont été conçus du point de vue mécanique. C'est ainsi que les ponts modernes sont devenus des installations mécaniques. Les problèmes cinématiques et dynamiques sont analysés.

ZUSAMMENFASSUNG
Auflager und Fahrbahnübergänge sind Bewegungsvorgängen unterworfen, für die sie in der Regel maschinenbaumässig konstruiert wurden. Damit erhalten moderne Brücken maschinenbaumässige Eigenschaften. Probleme kinematischer und dynamischer Art werden untersucht.

DESIGN, PERFORMANCE AND MAINTENANCE OF BEARINGS AND JOINTS

In the autumn of 1981 the American Concrete Institute held a World Congress on the above topic. When the one week Congress opened there were 36 lectures prepared and printed on 734 pages [1-0].

The subject matter is therefore extensive and must necessarily be limited here to a brief outline of a few aspects.

1) INTRODUCTION

Bearings and expansion joints present more problems for the civil engineer than the remaining construction components of bridges [2]. This observation made in 1971 could be repeated unchanged in 1981 [1-1].

Bearings and expansion joints are both attributes of modern bridge construction, which only entered the picture at a later date. They became necessary when stone arch bridges were superseded by beam type steel structures, which altered considerably in length with changes in temperature. The art of bridge building, however, dates back to antiquity as still revealed today by arch bridges, which have formed part of the landscape apparently unchanging for hundreds of years. And it is this very tradition which makes it difficult for the civil engineer to come to terms naturally with the kinematic and dynamic problems of bearings and expansion joints. We must, however, accept that the actual construction problem of bridges today must also include the techniques of mechanical engineering and plastic design.

Bearings and expansion joints are both subject to movement processes. This, however, is in fact all these two construction elements have in common. In function and design they are basically different.

The bearing transmits loads, which normally change very little, and performs specific displacements and rotational motions with these loads. This function is a static and kinematic one and easily identified. In addition, there are accurate conceptions of materials and function tests. It is therefore understandable that regulations and standards primarily concern the more simply defined problems of bearings [3], [4].

The expansion joint in a carriageway as a transition structure must be considered in a basically different way. While the bearing is relatively motionless under the bridge, the transition is constantly moving under dynamic loads. The kinematic function of closing the verying carriageway joint is made more difficult by the necessity of withstanding dynamic traffic loading. The transition structure should also be watertight - to protect the bearing. Insulating materials and surfaces must be connected and should also be concealed so ingeniously in the carriageway that road users are unaware of their presence. The problem is therefore extremely complex and is impractical to control by introducing regulations and standards.

Functional tests on carriageway transitions are not simple to perform and usually only possible as part of the overall function. They are, however, urgently necessary in order to provide a firm basis for specifying the minimum performance required to ensure a positive influence on further developments. Regulations only appearing in the form of restrictions can only hinder future developments.

Interesting proposals for function tests on carriageway transitions can be found in [1-2]. Tests on anchoring systems in reinforced concrete are published in [1-4] and [5], while [6] and [7] provide measurements on carriageway transition with traffic loading.

Bearings and carriageway transitions are subject to movement processes, for which they are normally mechanically designed. And since motion is synonymous with wear, the maintenance of a bridge is associated with problems similar for example to those in the servicing of a vehicle. The bridge tests prescribed at regular intervals take account of this, but the special nature of the bearings and carriageway transitions should be emphasised more particularly in the instructions in the sense described, in order to identify them as mechanical structures [8].

The maintenance of a bridge is intended to prevent wear. The replacement or restoration of parts subjected to wear must be made according to test results and plan. It is in principle an extremely familiar task, when it is appreciated that the asphalt surface of a bridge is also subjected to wear and must be renewed.

Design faults on the other hand are congenital defects, which can usually only be remedied by surgery. These are the actual causes of damage, which are informatively described for example in [1-1].

Bearings and carriageway transitions, therefore, have their own problems. But structural problems disappear if the construction itself is avoided. This simple statement ought to appear at the beginning of every bridge design. There are a number of interesting possibilities: slender piers can bend, concrete hinges can tilt and even carriageway transitions can be dispensed with if for example a bridge is curved in the form of a quarter circle [1-3]. But it is only in exceptional cases when the skilful civil engineer can design a bridge without bearings and expansion joints.

2) BRIDGE BEARINGS

Without going into the history of bridge bearings, which is outlined in practically all introductions to the subject, only the structures relevant today will be considered here under kinematic aspects. A full description of bearings and bearing systems is given in [9] and [10] .

2.1 Degrees of freedom and constraints

These are the factors with which bridge bearings can be clearly differentiated kinematically. They should automatically possess rotational possibilities in all directions these days, since this is absolutely essential for universal application. Point rotation is thus the kinematic basis of all modern bridge bearings.

For a long time, line rocker bearings and roller bearings were standard components in bridge construction. When resistance to movement was drastically reduced by hardened rollers, however, fractures of these sensitive bearing elements revealed previously ignored constraints [11]. These are to be expected if the superstructure bends in the transverse direction or is displaced obliquely to the direction of rolling. Bending moments are then produced in the bearing line for which a roller bearing cannot be designed. Roller bearings can therefore only be employed under clearly defined conditions.

Bridge bearings are designed to accept general forces and not to transfer moments. Their basis structure is simply too small for this purpose. Instead, they should offer definite degrees of freedom for rotation about all three axes. Point rocker bearings rotate about both horizontal axes, normally also about the vertical axis, but exceptions also have to be noted in this case.

In addition to the vertical loading, horizontal forces from both axial directions are also borne if the point rocker bearing is fixed. A point rocker bearing movable in one direction with three degrees of freedom of rotation also possesses one degree of freedom of displacement, and a bearing movable in all directions has two such degrees of freedom. In this case, only the degree of freedom of vertical displacement is restricted, in order to accept the vertical loading.

Guide bearings, however, which are not intended to accept vertical loads, logically possess the degree of freedom of vertical displacement and instead restrict one or both horizontal directions of displacement. Guide bearings remain special cases, since normally the load accepting bridge bearings can already provide the displacement direction or fixed point. But they should also be defined as point rocker bearings, which likewise should possess the three degrees of rotational freedom.

It is extremely useful and instructive to consider the design of a bridge bearing not only from the aspect of load acceptance and direction of displacement, but also to study it the other way round of whether degrees of freedom are restricted which do not have to be. For damage to bridge bearings can very often be attributed to rotations or displacements, which it was not necessary to restrict for the bearing function - these are termed constraints [12].

2.2 Reinforced elastomeric bearings

The reinforced elastomeric bearings, above all, must be considered in this light, since these can largely avoid the constraints by elastic resilience. The six degrees of freedom of spatial displacement and rotation are already provided by elastomer as a material property, owing to its ductility. In order to be useful as a bearing, its vertical ductility must be restricted or stiffened. With the pot-type bearing this is accomplished by enclosing the incompressible elastomer and with the elastomeric bearing by reinforcement usually with laminated steel, which subdivides the bearing block horizontally and therefore provides sufficient vertical stiffening for its flexing under load to remain negligible from the constructional engineering aspect [Fig. 1].

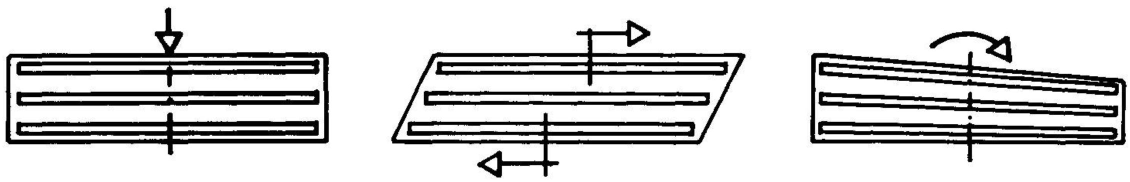


Fig. 1 Laminated Rubber bearing

The displacement capacity in all directions in relation to the elastomer depth is not restricted. This also applies to rotation about the vertical axis, while rotation about the two horizontal axes is indeed inhibited, but is useful owing to a corresponding elastomer depth. A general description in noteworthy detail of the reinforced elastomeric bearing is provided in [9]. Some new considerations and findings can be found in [1-0].

Reinforced elastomeric bearings are good-natured bearings, which if amply dimensioned absorb constraints without complaint.But they are also wilful bearings,which demand forces and moments and also the necessary possibility of movement for their deformations. Hence in a system of elastomeric bearings, the fixed point and also the direction of displacement are elastically variable and not capable of accurate determination. Bridge engineers should accept this as a desirable freedom for design and not make the error of trying to convert an elastomeric bearing to a fixed bearing, for example, by means of questionable auxiliary structures. If really necessary, it is simpler and cheaper with a steel rocker bearing, which also actually functions free from play in the required way [13].

Reinforced elastomeric bearings are simple and maintenancefree. They are important wherever their diverse degrees of freedom can be utilized, particularly in the low and medium range of loadings, displacements and rotations.

The displacement range of the reinforced elastomeric bearing can be extended with a sliding bearing mounted on top to obtain kinematically interesting shear -slide combinations. It should be noted in this case that a reinforced elastomeric bearing distributes the loadings over its area in the manner of a stress slope and manifests its rotational resistance in edge compressions of the PTFE-sliding disc. This makes the simple bearing somewhat more complicated [Fig. 2].

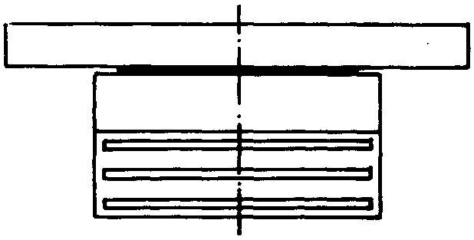


Fig. 2 Sliding Laminated Rubber Bearing

2.3 Point rocker bearings

Two building elements can mutually rotate or tilt by rolling together, sliding or deforming. Figs. 3 to 5 show these three typical forms of motion with examples of point rocker bearings, which can be combined in each case with sliding bearings. Various design principles with differing characteristics are involved, which are of importance in limiting conditions.

2.3.1 Steel rocker bearings

The steel rocker bearing shown in Fig. 3 rolls on a circular thrust member about both horizontal axes. The radii of curvature of the thrust member and possibly also of the bearing plate above are dependent on the loading and quality of the steel. They are normally so large that, on rotating the load moves noticeably from the centre. This is of special importance for the steel rocker sliding bearing, since the PTFE sliding disc is then loaded eccentrically. The permissiible mean PTFE compression cannot then be utilized, so that the sliding friction becomes higher than necessary [10]. The horizontal forces are absorbed without play, since the frictional tightness is normally adequate under the vertical compression and deformation. The stop of the bearing plate above also has an effect [13].

The rotations about the two horizontal axes are not thereby affected, but it should be realised that the frictional tightness inhibits rotation about the vertical axis. This is of little importance for the fixed steel rocker bearing and none at all for the steel rocker sliding bearing movable in all directions. It may however be necessary to correct the steel rocker sliding bearing movable in one direction by a rotation on the thrust member to compensate for an installation error in the direction of movement, without detriment to the sensitive sliding guide.

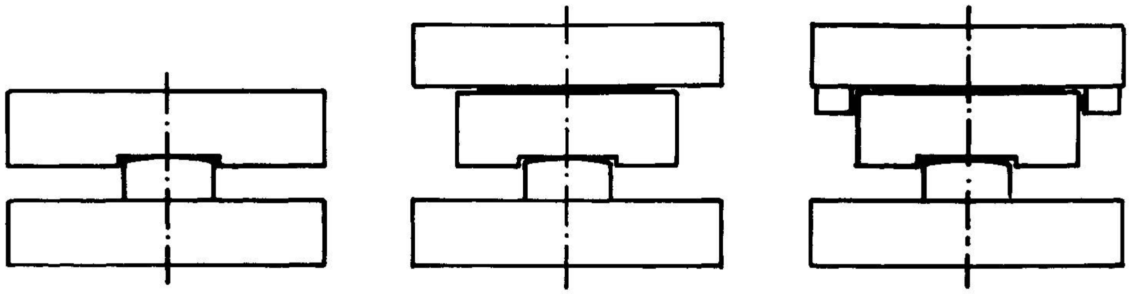


Fig. 3 Steel rocker bearing - fixed, multidirectional and unidirectional

From the contact point on the thrust member the load must be distributed over both bearing plate surfaces. This necessitates a large overall height, which is a hindrance to the transmission of large horizontal loads, since the edge compressions of the bearing plates and the PTFE sliding disc are correspondingly increased by the effect of moments [10]. The rotating part of the bearing is constructed in a remarkably simple and robust manner, but this desirable characteristic also influences the sensitive sliding piece.

2.3.2 Spherical bearings

The spherical bearing as shown in Fig. 4 rotates about both horizontal axes by means of the male portion, sliding cap in the concave lower bearing plate. The spherical radius of this rotating device is made as large as possible in order to obtain a roughly uniform PTFE compression in the curved sliding surface and to limit the thickness of the lower bearing plate [14].

The eccentricity of the applied load increases with the radius, however, since the rotational resistance effective in this case can be taken as the product of sliding friction and spherical radius [13]. In addition, the sliding friction must also be taken into account, which appears in the plain cap surface as a secondary load during rotation. An additional, geometrically caused eccentricity of the load, originating from the rotated position of the cap, is dependent on the overall height of the spherical bearing [10]. Various influences must therefore be considered in order to arrive at optimum rotating conditions.

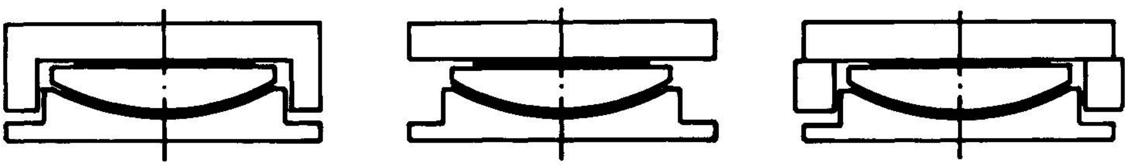


Fig. 4 Spherical Bearing - fixed, multidirectional and unidirectional

The curved sliding surface of the spherical bearing is not suitable for accepting horizontal loads. It would cause damage to the curved PTFE sliding-rotating disc placed within. For this reason, the upper bearing plate of a fixed spherical bearing is separated from the cap by a plain sliding surface. The horizontal forces are exerted directly on the lower bearing plate with a rotating stop. This system is not free from play and is subject to loading by the mutual friction of the stops during rotating motions.

It is clear from Fig. 4 that the basic form of the spherical bearing is of a bearing movable in all directions. The fixed spherical bearing is only produced from this by means of an auxiliary structure, comparable to the similar problem with the reinforced elastomeric bearings. This also applies to spherical sliding bearings movable in one direction, for which instead of a circular, surrounding stop, two guide rails couple the upper bearing plate with the lower certainly in sliding connection, but not without constraint, since the slide guides are loaded eccentrically when rotating about the axis in the sliding direction and these react more sensitively than the steel stop of the fixed bearing. Even more important, however, is the lack of freedom for vertical rotation. While on the steel rocker bearing it concerns restricting by friction in the same case, two rigid bearing plates are coupled together here. The function of the spherical bearing movable in one direction is dependent on the precise alignment between the slide guides of the bearing and the direction of displacement of the bridge deck [3]. Limiting conditions for movement of this kind are familiar with roller bearings.

The overall height of the spherical bearing is only one third of that for the steel rocker bearing, since the rotating device does not have to be concentrated at one point, but can be extended practically to the bearing whole surface. The spherical sliding bearing movable in all directions conforms best with the design principle, since no untoward horizontal forces have to be transmitted via the mechanism of the sliding rotating device.

2.3.3 Pot-type bearings

The pot-type bearing shown in Fig. 5 rotates about both horizontal axes by means of the pot top plate acting as upper bearing plate deforming the elastomeric pad in the pot. This behaves like a viscous, incompressible fluid. The elastomeric is conveniently produced from natural rubber, which exhibits a uniform deformability over a wide range of temperature [11]. The elastomeric pad is also lubricated, so that in this condition the deformation from rotating takes places largely free from constraint. The eccentricities from the rotational resistance are negligibly small if the elastomer is sufficiently thick [14]. Dimensioning is based, however, on the dry or non-lubricated elastomeric pad, which perhaps occurs after a long time. This may probably only be of interest for rotation due to traffic loading.

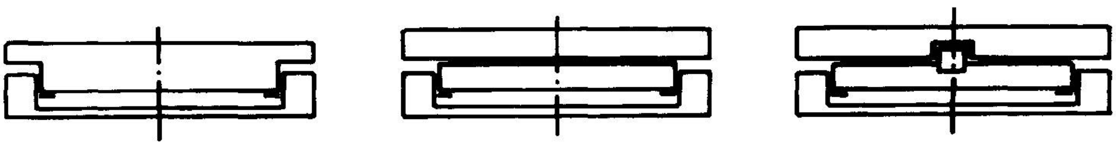


Fig. 5 Pot-type Bearing - fixed, multidirectional and unidirectional

The prerequisite function of the pot-type bearing is for a reliable sealing of the elastomeric pad against the movement gap between top plate and pot ring. The rotating part of the bearing must also be sealed externally, so that the elastomeric pad is protected and remains in the lubricated condition. The top plate located to the pot transmits the horizontal forces to the pot. As for the spherical bearing, the process is not free from play, but the surfaces in contact lie in the area of the grease deposit of the elastomeric pad, which extends in the movement gap up as far as the seal.

The functional connection between spherical and pot-type bearings is most clearly apparent with the bearings movable in all directions. Rotation and sliding are very similar in effect in both types. One basic difference can be seen, however, with the bearing movable in one direction. The pot-type bearing possesses a definite degree of freedom for rotation about the vertical axis, since rotating and sliding parts are clearly separated. It can rotate almost without resistance on the lubricated or even dry elastomeric pad, while this is only possible to a limited extent with steel rocker bearings and not at all with spherical bearings.

The overall height of the pot-type bearing is generally similar to that for the spherical bearing. It is important to know with respect to the sliding element that the elastomeric pad acting like a viscous fluid guarantees the required uniform compression of the PTFE sliding disc even with large tolerances in and under the pot. A disadvantage is the central guidance shown in Fig. 5 of the bearing movable in one direction, since the groove in the sliding plate represents a notch, similar to the guiding groove by roller bearings, which can lead to deformation of the sliding plate and to unacceptable compression of the PTFE sliding disc.

From the necessarily brief consideration of three different types of point rocker bearings, it is clear that there are significant differences even with respect to the degrees of freedom and constraints:

- With the steel rocker bearing the fixed bearing is of incomparable simplicity and robustness. It is an all-steel design, with the sole disadvantage of large overall height. The movable bearings are unsatisfactory.
- With the spherical bearing the type which is movable in all directions is a successful design, while the version movable in one direction has inadequate degrees of freedom.
- Pot-type bearings display no kinematic faults. Fixed bearings and bearings movable in one or all directions possess all degrees of freedom of rotation and displacement in all cases, provided they do not have to be restrained for a guiding purpose.

2.4 Design, performance and maintenance of a bridge bearing

All the relevant details have not been settled when the type of bearing has been decided for a structure, since the same design principle can be implemented in many different ways even within tight regulations. The construction, function and maintenance will be described with the example of a carefully designed pot-type bearing movable in one direction, to demonstrate their importance for the life of a structure [Fig. 6].

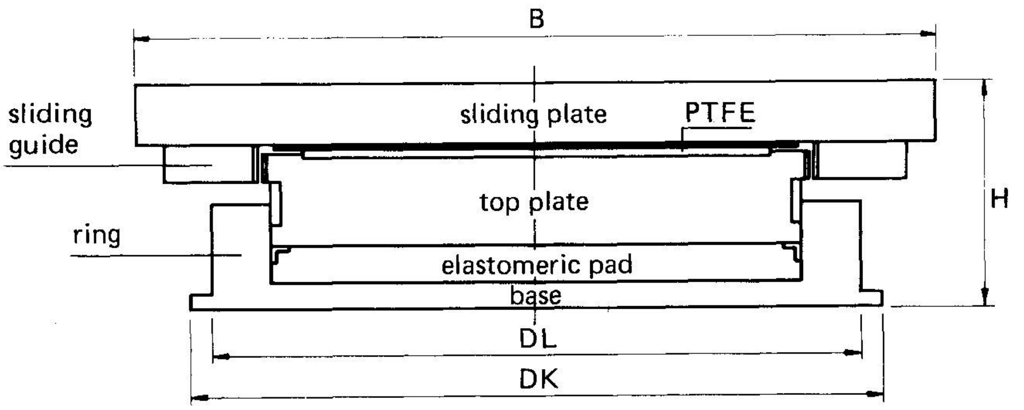


Fig. 6 Unidirectional Sliding Pot Bearing

The most important detail of a pot-type bearing is the sealing of the elastomeric pad against the movement gap between pot ring and top plate. Since the introduction of the pot-type bearing in 1959, three brass rings lying one above the other have successfully been used for this purpose. These rings are placed in a surrounding groove of the elastomeric pad so that their joints overlap [14]. In the search for materials capable of sliding better, a dimensionally stable sealing chain was developed, which consists of interlinked polyoxymethylene members (POM). It is vulcanized in at the time of manufacture of the elastomeric pad, so that it is immovably fixed in the pad when the bearing is assembled. Fig. 7 shows an elastomeric pad with this sealing chain, which slides without wear on the wall of the pot and adapts easily to all deformations of the pad owing to its chain-like structure.

The elastomeric pad must remain flexible in cold conditions. This requirement is not only limited to the rotational resistance, but also in particular to the security of the pad sealing, which would be subject to unacceptable loading from stiffening elastomer. An elastomeric pad of natural rubber avoids this disadvantage. It is not only unaffected by all ageing influences in the pot by the compressed gap seal, but also reliably protected by a further grease seal incorporated in the edge of the top plate [11].

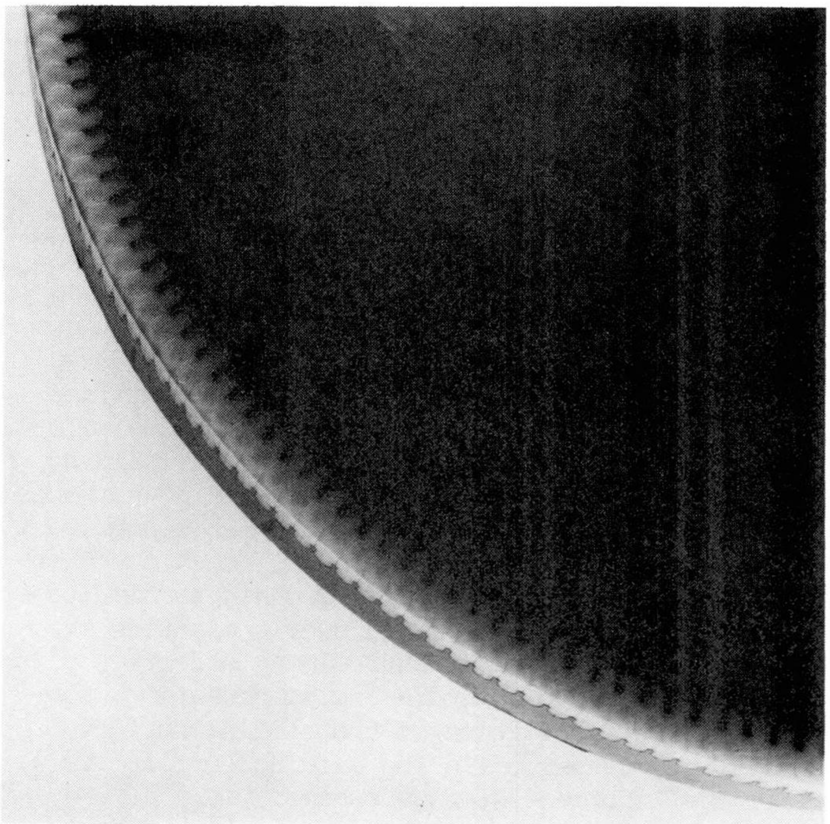


Fig. 7 Rubber pad with sealing chain

The elastomeric pad lies greased in its pot and during rotating motions slides on its grease bolster, which should always be present between pad and steel surfaces. Elastomeric pad, gap seal and sliding grease form a closed system of three elements, which must prove its reliability by joint endurance tests on a large scale before it can be responsibly employed [15] , [16] .

This also applies to the sliding bearing mounted on the top plate. Its three working elements are the PTFE sliding material, the high-grade steel sliding surface and the lubricant. The PTFE sliding material projects as a disc locked in a flat housing. The PTFE sliding disc should be sufficiently thick to provide a uniform supporting behaviour by plastic deformation. In conjunction with the requirement for limited edge compression from horizontal forces and rotational moments, this is a precondition for optimum sliding bearing design. This deformation capacity of the PTFE sliding disc is also utilized in order to achieve long-term lubrication of the system with lubricating pockets open to the sliding surface. In contrast with the elastomeric pad, the sliding disc of PTFE is exposed to destructive wear if the lubrication fails [17].

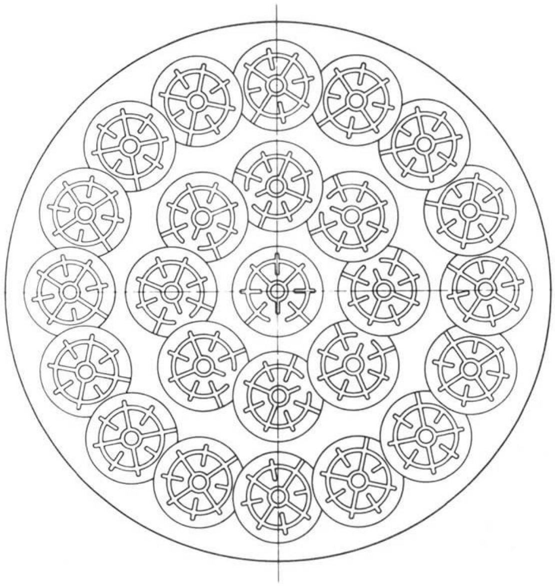


Fig. 8 Sliding Chain with Grease Storages

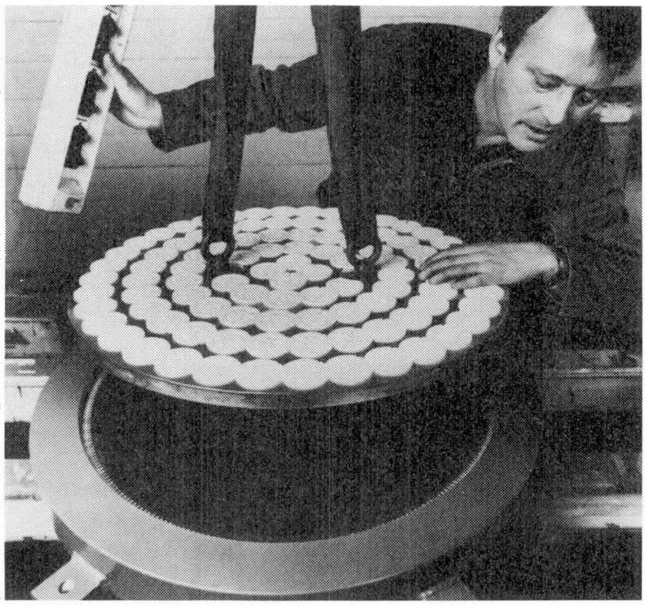


Fig. 9 Assembly of a Sliding Pot Bearing

This problem of long-term lubrication was solved in a new way with a modified sliding system. The enclosed PTFE sliding disc is divided into individual elements, each engaged in its own recess. They form intermeshing annular sliding chains, which like the annual rings of a tree can grow to form sliding surfaces of any size. Fig. 8 shows an arrangement of this kind, in which large lubricant storage spaces between the individual sliding chains ensure prolonged functioning of the sliding bearing free from wear. The outer sliding chain is simultaneously the sealing ring, so that by hydraulic compensation over the entire sliding surface the system acts as lubricating pad with particularly effective sliding characteristics [18]. This useful property of hydraulic compensation, however, is of the utmost importance for re-lubrication of the sliding surface, since fresh lubricant can very easily be inserted in the sliding surface via a hole leading from externally into the lubricant storage spaces.

In the sliding pot-type bearings shown in Fig. 6 the slide guides for movement in one direction are situated externally. This design is technically superior to the grooved central slide guide shown in Fig. 5. In addition to interference with load transmission, the notch in the sliding plate could possibly be a reason for questioning its flatness, particularly where substantial weights of fresh concrete may be expected during concreting of the bridge deck [1-5]. Damage to the sliding bearing by unacceptable PTFE-compression cannot then be excluded.

A carefully designed bridge bearing should also be clearly and unambiguously designated, since damage to bridge bearings is often due to installation errors pre-programmed by inadequate identification.

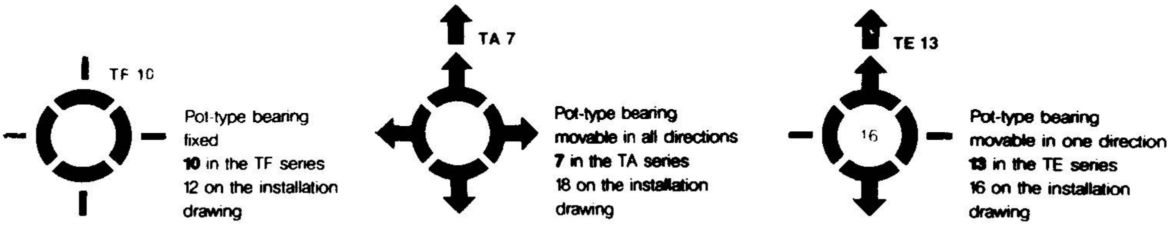


Fig. 10 Captions on Top Surface of Bearing

When placing a bearing attention is given to the upper surface. Everything should be marked there of importance for installation. Fig. 10 shows clear markings and their significance for fixed bearings and those movable in one or all directions. The double arrow points towards the fixed bearing.

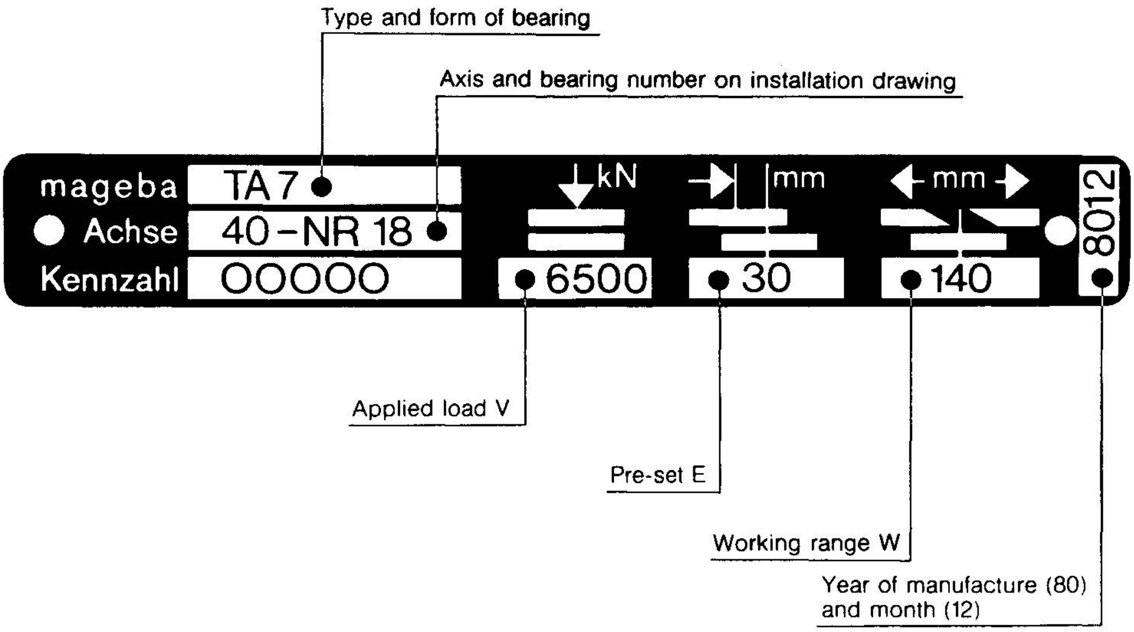


Fig. 11 Typeplate for Bridge Bearing

A typeplate should accompany every bridge bearing as a permanent and externally visible identification card. A typeplate of this kind is shown in Fig. 11 for a pot-type bearing movable in all directions with explanation of all data.

A working scale as shown in Fig. 12 should be attached for operation of the bearing, indicating the pre-set and working range in addition to the direction towards the fixed bearing. The scale and its limiting values marked by arrows and the pointer position should easily be recognizable with field glasses during inspection.

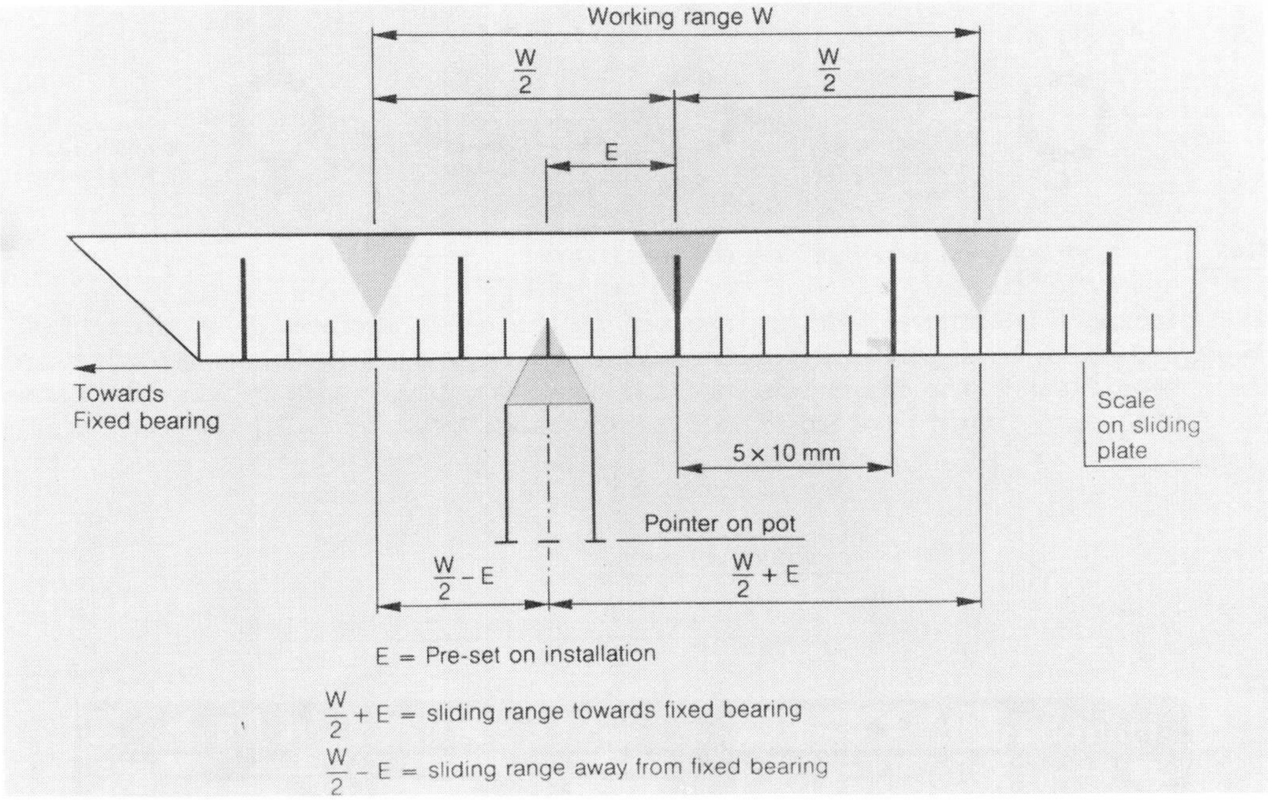


Fig. 12 Working Scale for movable Bearings

From the comments made above it is clear that bridge bearings are items which cannot be defined by the methods of building bridges alone. They are indeed designed in accordance with aspects of statics like the elements of a bridge, but their function is to accept loads in conjunction with the kinematic requirement for certain degrees of freedom. Apart from the smaller range of reinforced elastomeric bearings, this is only possible by mechanical engineering methods. For the maintenance of a bridge it follows from this that a bridge bearing also has parts subject to wear in addition to components requiring attention. It must be possible to replace these worn parts, since they risk losing their substance owing to their allotted function.

This would for example be the case for the PTFE sliding disc, if the lubrication failed. As the detailed description of the sliding pot-type bearing movable in one direction indicates, however, an endeavour is made further to develop the so-called wearing parts into components requiring care, which no longer lose their substance. But even highly encouraging results must still be proved by long experience. The principle formerly applied is that every bridge bearing must be replaceable at an acceptable cost.

3) EXPANSION JOINTS

Following the portrayal of bridge bearings, the mounting and suspension of expansion joints will be described as an important partial problem in the design of lamella joints. Comprehensive documentation on carriageway transitions and the associated problems can be found in [19].

3.1 Mounting and suspension

The kinematic function of bridge bearings can be clearly defined with the conceptions of degrees of freedom and constraints. For carriageway transitions with movable elements these terms are no longer sufficient to describe the scope of their movement functions. For it is not a question of accepting large loads mainly consisting of the dead weight with a bearing, but of mounting a structure movable in itself, which is more like a moving bridge than a bearing. An expansion joint has a number of bearings like a bridge, but it is exposed to the heaviest traffic loading nearly without dead weight. From this aspect, its basic function of closing the varying main gap in the carriageway becomes of secondary importance.

Since motions and simultaneous dynamic loadings are involved, mounting and suspension are the most suitable terms for describing this function. This can more readily be appreciated if one stands beneath an expansion joint and experiences the effect of the traffic roaring across. The effect is quite terrifying.

While the bearing in a bridge is already a different kind of design, the same is true in far greater measure for the expansion joint. The design principle should not only be mechanically oriented, but also pay far more attention to the mounting and suspension of a heavy vehicle, since in practice the expansion joint is the passive counterpart mounted in the bridge to the heavy vehicle rolling across.

The numerous old carriageway transition structures in our bridges were naturally constructed in earlier days with different concepts of design. They are scarcely equal to the demands made by modern heavy traffic and will largely have to be replaced in the long-term.

Mounting problems are encountered with all designs, which close the main gap with plate-like elements. These are the old cover plates on two bearing lines or the rocker plates on four bearing cams. They must be so stiff because of the wheel pressures from heavy vehicles that even powerful spring forces cannot adapt them to the statically indeterminate mounting. They shake and begin to knock under the traffic, making audible the incipient wear. Nor can this problem be avoided by complicated chain plates.

3.2 Resilient Mounting of Lamella Joints

The design of a lamella joint is made clear in the sectional model in Fig. 13. The lamella are grille bars, which unlike the familiar finger grilles do not intermesh, but divide the expansion path in traversable individual gaps in the direction of the traffic. This has the initial important advantage of simple sealing with elastomeric expansion sections, as is customary with only one joint gap with the small expansion joints.

Further advantages of the lamella design are revealed when the mounting is examined. Each lamella is secured against tilting, since it is firmly connected to its own cross-bearers, which provide it with a bearing base of joint width. This bearing base, however, not only signifies tilting security, but also a lever arm at the same time with which varying bearing conditions can easily be corrected by means of the torsional elasticity of the lamella. And this is possible without having to restrict the bending resistance of the lamella required for loadbearing.

Herein lies the basic difference in the mounting of lamella and stiff rocker plates.

This correction of the bearing conditions is enforced by the pre-tensioned sliding springs at each end of the cross-bearer. At the same time, they ensure a positive contact with the sliding bearing, which accepts the cushioning and damping of the wheel loadings with its resilience.

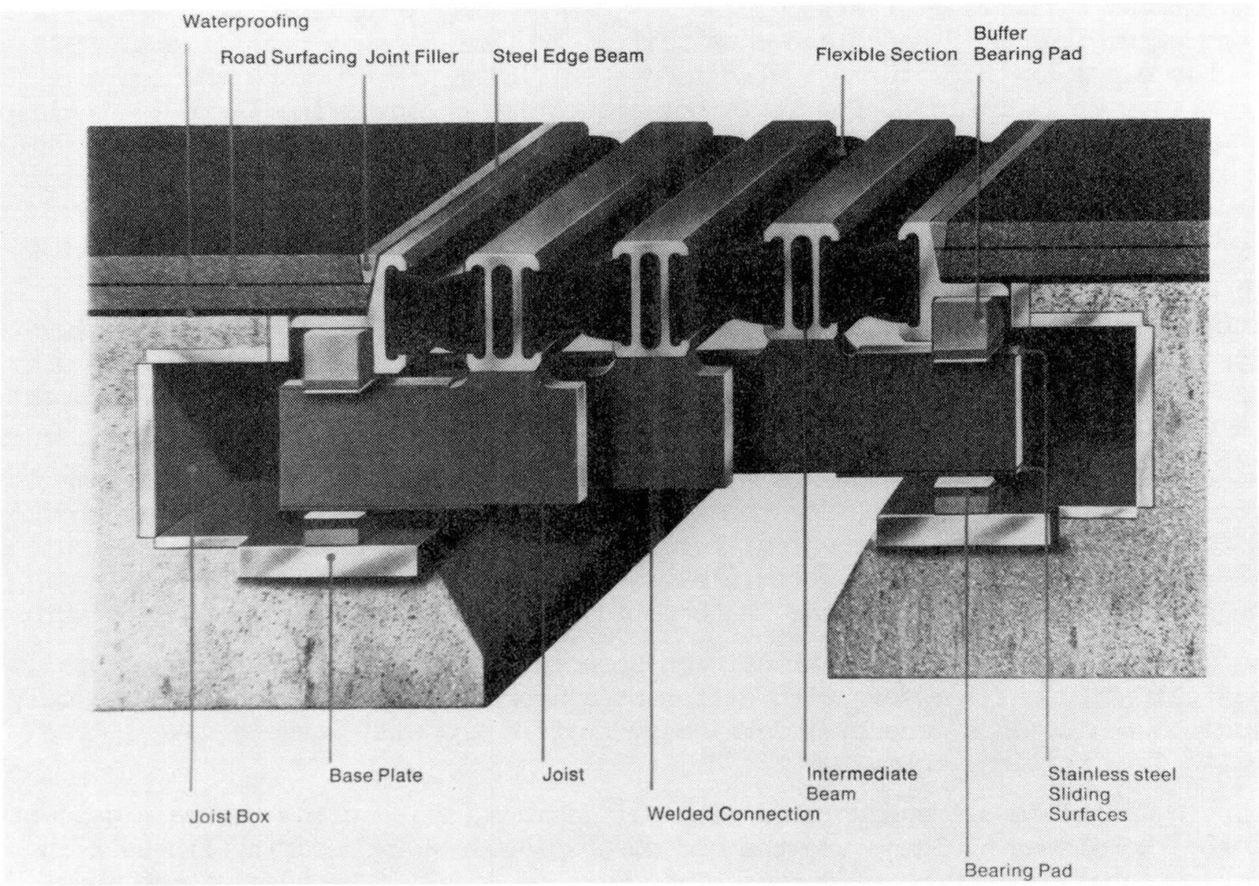


Fig. 13 Model Section of Lamella Joint

The diagram of Fig. 14 illustrates the qualitative correlations of the pre-stressed resilient mounting of a lamella joint in accordance with the sectional model of Fig. 13. The force P is plotted along the Y-axis against the flexing f along the X-axis. The steeper working line of the sliding bearing A_L starts at the origin, while the flat one of the sliding spring A_F begins where the overall system is sprung with f_S on assembly. The point of intersection S of the two working lines gives the pre-stressing value P_V and the division of the flexing f_S in the smaller sliding bearing section f_L and in the larger sliding spring section f_F. The cross-hatched triangle in the diagram marks the working area of the pre-stressed and resilient mounting. An external force P_A initially accepts the pre-stress value P_V of the sliding spring with the part P_1, until with the part P_2 it causes a further flexing f_A of the sliding bearing and a back-springing of the sliding spring.

Smaller wheel loads therefore traverse the expansion joint without flexing. Only with greater loadings from heavy traffic does the system provide cushioning and dampening in the desired manner, whereby the pre-stressing and spring stiffness are adapted to the upper limit of the load.

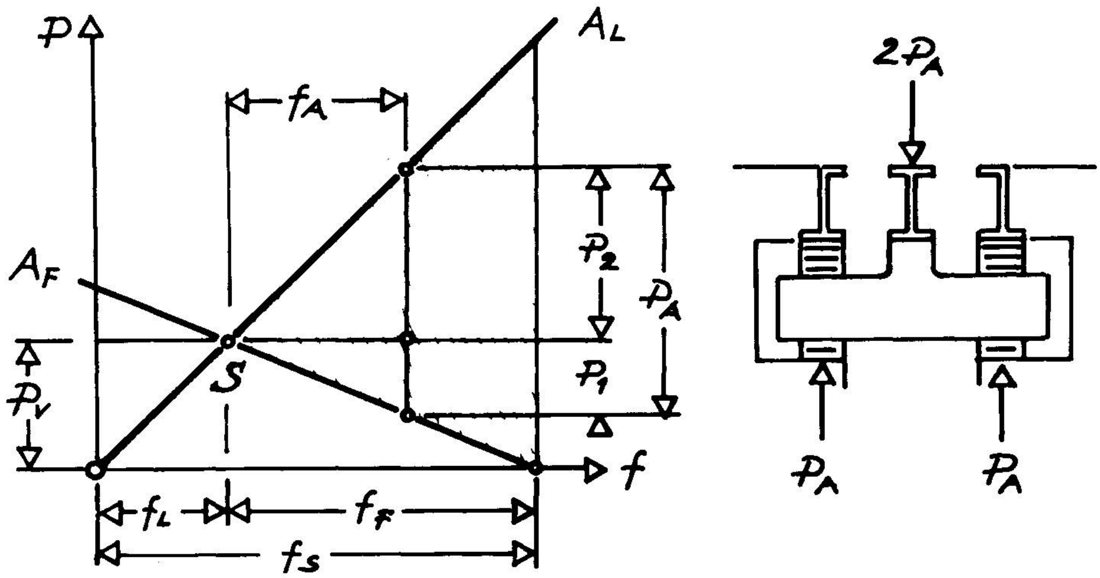


Fig. 14 Diagram of Resilient Mounting System

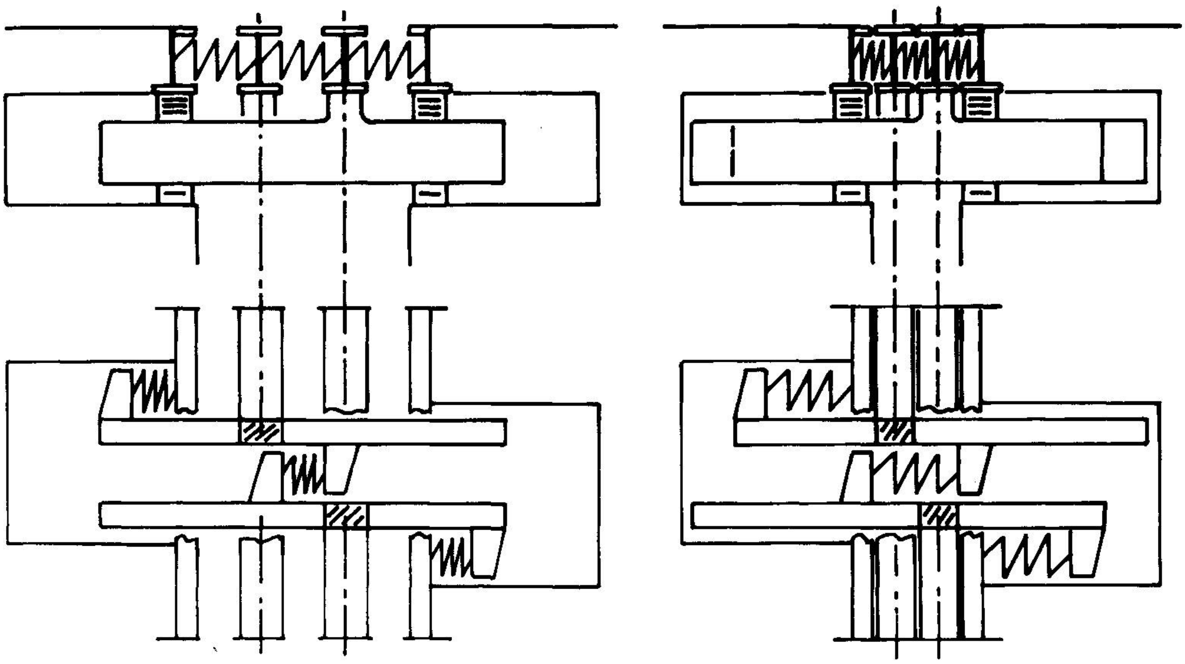


Fig. 15 Resilient Dual Control System

3.3 Resilient Control of Lamella Joints

Expansion joints are both vertically and horizontally loaded. Braking and accelerating forces are applied by the traffic and resistance of displacement is provided by the structure itself. These stresses necessitate control of the individually movable lamella to form a kinematically co-ordinated overall system.

This function can similarly be solved by means of resilient pre-stressing. And this is particularly effective when two pre-stressing systems work in opposition. Fig. 15 shows a cross-section and plan of a lamella joint in both the open and closed positions of the joint with a dual pre-stressing system of this kind. The cross-sections show the expansion sections represented by spring symbols as one system, while the plans depict control springs with same symbols as the other system. One operates between the lamellas, whilst the other works in the opposite way between the cams of the cross-bearers. It can be seen that the two systems work in opposition when the joint opens and closes, whereby both systems intermesh via the connection from lamella to cross-bearer shown hatched in the plan view.

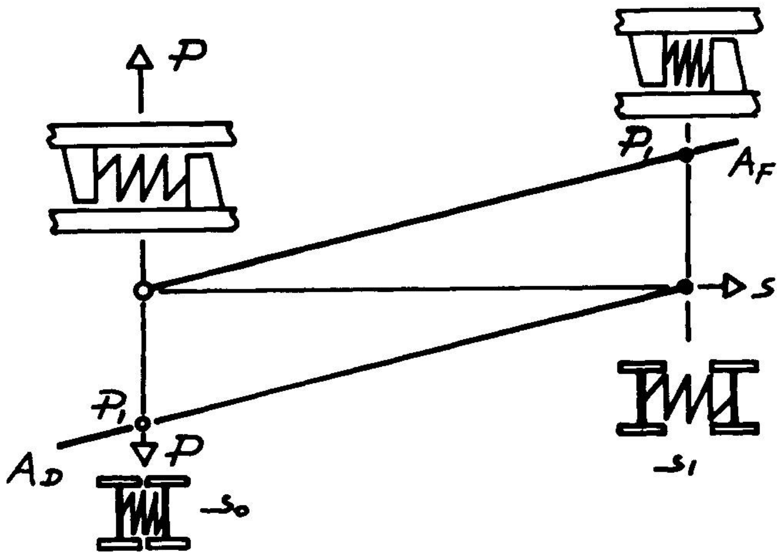
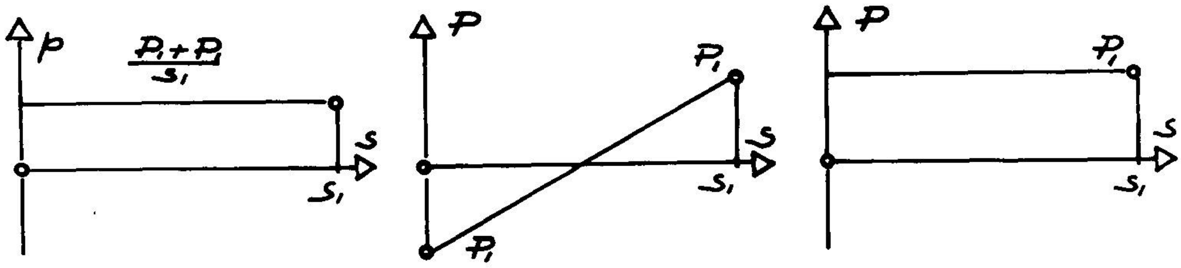


Fig. 16 Diagram of Forces for a Resilient Dual Control System



Control Force | Reaction Force | Deflection Force

Fig. 17 Correlations of a Resilient Dual Control System

The diagrams of Figs. 16 and 17 show the qualitative correlations. It is assumed for the sake of simplicity that both pre-stressing systems have the same ultimate force P_1 and linear working characteristics. In all diagrams the force is plotted along the y-axis against the joint position along the x-axis. The closed joint is indicated by S_0 and the open joint by S_1 at the origin of co-ordinates. Three effects of the resulting forces can be observed from the diagram of pre-stressing forces:

- The controlling force is the change in pre-stressing forces. It becomes effective when the lamellas have to follow on opening or closing of the joint.
- The reaction force is the difference of the pre-stressing forces. It is the resultant force applied from the system to the joint edges.
- The deflection force is the sum of the pre-stressing forces. It appears as a lamella resistance when this is deflected by external force such as braking or acceleration.

The noteworthy result is that both the controlling force and the deflecting force remain unchanged in all joint positions and that in addition the maximum value of the reaction force is only equal to that of one pre-stressing system. It becomes zero at the central position and applies no load to the bridge deck.

More attention must be paid to the avoidance of constraints with expansion joints than with bridge bearings, since the expansion joint in the carriageway is far more exposed to uncontrollable movements owing to its exposed position and spatial area than a defined bearing point below the bridge deck. The expansion joint must as it were always be a bearing movable in all directions with all degrees of freedom of rotation and displacement. Only for load compensation is the vertical displacement restricted, but under the dynamic conditions of cushioning and dampening. The lamella joint described with a grille bar system consisting of lamella and cross-bearers, together with resilient pre-stressed mounting and dual control, is equal to these demands.

There are also other solutions, however, for lamella joints, which instead of resilient elements employ kinematically more precise devices. These are, for example, lever or shear structures to control lamella and with which their loading can also sometimes be borne [20]. The attraction of such designs lies in the exact synchronization of the lamellas, which - if successful - certainly looks favourable, but must be obtained at the expense of a kinematically limiting mechanism. The comparison is raised here between the deformation possibilities of reinforced elastomeric bearings and those of steel bearing structures.

Mechanical control devices as primary system are important, however, when resiliently controlled lamella joints comprise more than ten or twelve lamellas. They are thereby divided into groups and thus permit expansion paths of any size in watertight lamella structures.

4) SUMMARY

Bearings and carriageway transition structures can be assessed by means of simple kinematic and dynamic considerations without mathematical calculations, in order to compare various designs. This fact should enable the civil engineer to understand the problems of bearings and carriageway transitions and to solve them in a technically satisfactory way. The success of a structure - and this is above all the duration of its usefulness - largely depends on this.

During the past two or three decades we have experienced an extremely rapid development. This has taken us so to speak from medieval time to contemporary times, if we consider the days without bearings and expansion joints as antiquity. Today it is a matter of consolidating current achievements, since spectacular new developments are hardly likely in the near future.

Consolidation of the field includes the functional tests already mentioned in the introduction, which are only in their infancy with respect to structures for expansion joints. For bearings they are already covered by regulations similar to standards, as for example is shown by the approvals for bridge bearings in Germany.

These findings from bearing tests are also in fact used for structures ten times larger than the test specimens. But it is perfectly normal in technology, however, that familiar designs also reveal previously concealed problems in a new order of magnitude. And unfortunately the technical order of magnitude is also associated with a financial order of magnitude.

It is certain that bearing designs cannot be extrapolated at will, even if contrary statements have a reassuring effect [1-5]. The reason for an opinion of this kind is simply because corresponding testing equipment has never existed. Reference may be made here, however, to a new testing device in Switzerland. [1-6] and Fig. 18. Loads up to 100'000 kN can be applied for testing bridge bearings. All the functions of a bridge bearing are carried out under this applied load and under additional horizontal forces for the various displacements and rotations.

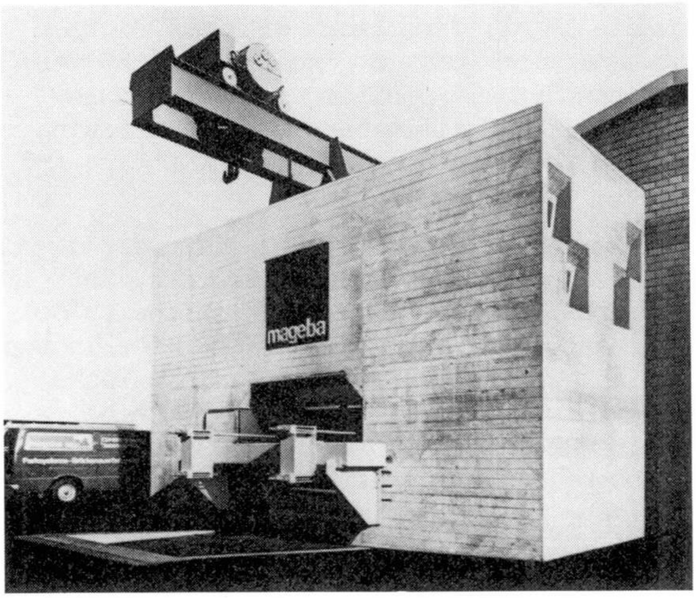


Fig. 18 Test Rig for Loads up to 100'000 kN

NOTATIONS

The figures have been provided by:

Mageba SA, Bülach, Schweiz: 6, 7, 8, 9, 10, 11, 12, 18

Maurer Söhne, München, Deutschland: 13

Author: 1, 2, 3, 4, 5, 14, 15, 16, 17

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Bearings and Expansion Joints for Bridges

Appuis et joints de dilatation des ponts

Auflager und Dilatationsfugen bei Brücken

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SUMMARY
The paper reviews recent experience in the United Kingdom and overseas relating to trends in theory and practice of bearing and expansion joint design for bridges. References will be made to the new Specification for Bridge Bearings embodied in the new British Standard BS 5400: Part 9, Bridge Bearings. This is the first British Standard to cover the major types of bearings used in Bridges in one document. The importance of installation of bearings and expansion joints will be emphasised and in relation to experiences of the author over the last twenty years.

RESUME
Le document traite d'expériences théoriques et pratiques en Grande-Bretagne et outre-mer en relation avec l'évolution des appuis et joints de dilatation des ponts. De nouvelles règles pour des appuis de ponts sont incorporées dans les nouvelles recommandations britanniques BS 5400: partie 9, appuis de ponts. C'est la première recommandation britannique qui couvre, en un seul document les principaux appuis de ponts. L'importance d'une installation d'appuis et de joints de dilatation est illustrée sur la base d'expériences faites par l'auteur dans les vingt dernières années.

ZUSAMMENFASSUNG
Der Bericht behandelt neuere Erfahrungen in Grossbritannien und in Übersee bezüglich der Planung von Auflager und Dilatationsfugen für Brücken. Es handelt sich dabei um eine neue Spezifizierung für Brückenauflager, welche in der neuen britischen Norm BS 5400 verankert ist: Teil 9, Brückenauflager. Dies ist die erste britische Norm, welche die hauptsächlichen Auflagertypen die im Brückenbau verwendet werden in einem einzigen Dokument zusammenfasst. Die Wichtigkeit der Installation von Auflagern und Bewegungsfugen wird aufgrund der zwanzigjährigen Erfahrung des Autors hervorgehoben.

1. TRENDS IN THEORY AND PRACTICE

1.1

In the Introduction to his book published in 1971 [1] the author stated that bearings and expansion joints caused more difficulties for the bridge engineer than practically any other part of the structure. The experience of the decade since that was written has not served to alter that view.

1.2

There is little doubt that bearings with a minimum of moving parts perform well. The designs evolved in the last 30 years, such as the laminated and contained elastomeric bearings, together with the use of the PTFE sliding interface types have proved a notable development of great value to the bridge enigneer. The development of multi-span bridges fully continuous for distances up to one mile has been possible by the use of sliding bearings. Such structures have demonstrated good riding qualities and low maintenance because the continuity over the piers has simplified the protection of the bearings against ingress of dirt and moisture from the road deck. The types of elevated roads with curved alignments and complex junctions have stimulated the design of robust bearings able to accommodate movement and rotation in both longitudinal and lateral directions.

1.3

There is little doubt that good design and manufacture of bearings and expansion joints is required but just as important is adequacy of installation and sound construction techniques for both the sub-structure and the super-structure. There are the majority of examples where performance of structures is excellent with a trouble-free service life.

1.4

Nevertheless cases have arisen where the author's firm has been called in by the Client to investigate problems stemming both from inadequacies in the bearings themselves and deficiencies in the bridge structure around the bearing. These can be extremely difficult to deal with, especially on heavily trafficked roads, and the cost of repair and maintenance can be very high in proportion to the initial cost of the bridge. These matters will be reviewed later in this paper but from the theoretical point of view, whilst the trend towards heavy loads and continuity of structure has been beneficial, it must be allied to first class detailing by the engineer to ensure that his intensions are fully realised on site.

2. SPECIFICATION OF BRIDGE BEARINGS

2.1

Up to the middle of this century bridges relied on roller, rocker or metal sliding bearings to permit movement. With more advanced designs to make full use of the materials employed and increased use of skewed and curved bridges to carry modern high speed roads over obstructions, the need arose for bearings to take movement in more than one direction. New types of bearings have been developed taking advantage of the new materials arising from improved technology. No doubt others will be developed in the future but it will be necessary to ensure that they are at least as reliable as those already in service.

2.2

BS 5400 : Part 9 is the first British Standard dealing comprehensively with the design, manufacture and installation of bearings for steel, concrete and composite bridges. It does not cover concrete hinges, nor bearings for moving bridges (e.g. swing and lift bridges). Also it does not include bearings made with proprietary materials such as Fabreeka and Bonafy but provision is made for the use of such materials, provided the Engineer is satisfied as to their long term suitability for the function intended. The document is split into two sections; Part 9.1 is a Code of Practice and gives rules for the design of bearings. Part 9.2 specifies the materials, method of manufacture and installation of bearings.

2.3

The design section is written in limit state terms as used throughout BS 5400. The terminology used for differing types of bearing is defined. This is necessary as Engineers tend to use bearing terminology loosely which can give rise to confusion as to what is really meant when referring to bearings by named type. The design section gives an overall framework in which the bearings are to be used and then deals with specific requirements for the various types of bearing and bearing materials.

2.4

Information in respect of frictional resistance of PTFE sliding surfaces has been brought up to date and the effect of temperature on the stiffening of elastomer has been recognised. The permitted shear strain of elastomeric bearings under horizontal movement has been increased to 0.7 and the method of their design generally brought in line with the U.I.C. Code [3].

2.5

The one area in which it has not been possible to give much detailed advice is in connection with pot bearings, the effectiveness of which is largely dependent upon the seal preventing the rubber from extruding between the piston and pot wall. The design expertise for such bearings is mainly in the hands of specialist bearing manufacturers.

2.6

Because of the high friction values associated with metal to metal sliding surfaces and complete seizure if not kept lubricated or corrosion is not prevented, modern sliding bearings usually rely on PTFE or similar low friction non corroding synthetic materials to provide the sliding surface. This has been recognised in the new Code and there is no information given on friction coefficient values or bearing stresses for metal to metal contact other than for guides.

2.7

In general, rolling, rocking and sliding surfaces have to meet the serviceability limit requirements, whereas the main supporting structure of bearings has to be designed for the ultimate limit state as set out in Part 1 of BS 5400 [4].

2.8

The Specification section covers the materials and manufacturing process most commonly employed in the manufacture of present day bearings. Quality control procedures have not been covered but performance tests have been outlined. It is hoped that a suitable quality control procedure can be set up in conjunction with the Agrement Board. Essential installation requirements have been laid down in the specification section.

3. LESSONS OF EXPERIENCE WITH BEARINGS

3.1

Bearing failure can result from a number of causes, e.g. damage or displacement following an accident, attack by chemicals, fire, corrosion of contact surfaces, but probably the greatest cause of bearing malfunction, particularly of modern bearings, is due to inadequate or improper installation. It is not unknown for bearings to be installed upside down. It cannot be stressed too strongly that care in the installation of bearings is of the utmost importance.

3.2

Bridges are usually designed with an expected life in excess of 100 years [4]. Modern bearings and bearing materials have not been proved in service for this length of time so it is advisable to make provision in the design of bridges for bearing replacement should this be found to be necessary. Facilities for correcting the effects of differential settlements, etc., should be provided unless the structure has been designed to accommodate such movements.

3.3

Regular inspection of bearings should be made so that any potential trouble is detected before serious damage is done to the structure. There should be adequate space around bearings to allow for inspection and maintenance in service. In certain circumstances, such as when piers or abutments are high or over water, it may be advisable to incorporate some form of travelling staging in the bridge design to facilitate inspection.

3.4

To ensure that the moving surfaces are not contaminated it is essential that bearings are not dismantled after leaving the manufacturer's works. Although steel bearings may be bolted directly to steel structures or steel plates cast into concrete structures, provided that they are within the tolerances required for the bearings, it is usual to lay bearings on some form of bedding. Commonly used materials are cementitious or chemical resin mortar, grout or dry packing [5]. It is essential that any bedding material, whether above or below the bearing, extends over the whole area of the bearing. It is also important that there are no hard spots and to this end any temporary packing used during erection of the bridge deck should be removed and the voids filled with bedding material.

3.5

The choice of bedding material is influenced by the method of installing the bearings, the size of gap to be filled, the strength and setting time required, and the composition and workability of the bedding material must be specified with those criteria in mind. Formwork should be sealed around bearings and the bearings, particularly the working surfaces, protected against grout leakage during the casting of insitu concrete bridge decks. Top plates should be supported and care taken not to displace or distort bearings during the concreting operations.

3.6

Evidence so far indicates that if designed and installed properly, modern bearings perform satisfactorily in service. Some problems encountered are outlined below:

3.7

Elastomeric bearings will take a considerable amount of maltreatment before failure unless grossly inferior materials are used. However, localised overloading due, for example, to uneven seating can cause breakdown of the bond between elastomeric and steel reinforcing plates. Unreinforced elastomeric strips can squeeze or work their way out under certain circumstances. Small seating plinths can disintegrate under shear forces generated by elastomer bearings and the seatings should extend at least 50mm beyond the edge of the bearing.

3.8

Disintegration of poorly prepared seatings is one of the most common causes of bearing failure. This problem has recently been highlighted at the Gravelly Hill motorway inter-change outside Birmingham, England. Here the bearing seatings have disintegrated and allowed the deck support beams to drop causing tension cracks in the locally unsupported deck slab [6]. (Fig 1a and Fig 1b)

3.9

At another project it is thought that incorrectly proportioned constituents (too much hardener plus a small quantity of water in the aggregate) led to the failure of 2" high epoxy resin bearing plinths when the precast concrete beams were lowered onto the bearings.

3.10

Incorrect installation procedures led to the failure of bearings supporting a viaduct over a river estuary. Here, large mechanical bearings were to be set on 12mm thick pads of polyester resin mortar with a sheet of polythene placed on top of the mortar bed to break the bond between the bearing and the mortar. The mortar was domed, the intention being that surplus material would squeeze out when the fixing bolts were tightened down. In practice the large quantity of resin mortar needed for each bearing required that it be made up in a number of mixes and consequently the material could not be considered as entirely homogeneous. On removing the damaged bearings it was found that the polythene sheeting had rucked. Both these results led to a non-uniform support to the bearing causing failure.

3.11

Leaking expansion joints can lead to corrosion of metal bearings. Unsuitable materials can give rise to problems. Many of the 18500 sliding rocker bearings installed in the Midland Links viaduct are not functioning as they should. The bearings are made of three rolled steel plates, the middle one heavily chamfered to allow the top plate to rotate. The steel deck beams rest directly on the top plate with no special sliding medium at the steel to steel interface apart from an initial coating of molybdenum disulphide. Some of the bearings have seized and those that still slide do so very reluctantly. Attempts to introduct lubricant between the sliding surfaces have proved ineffective [7]. (Fig 1a)

3.12

In a similar manner, the steel deck beams of Vauxhall Bridge over the River

Thames in London, built about 1906, rested directly on steel plates bedded on cill stones. Over the years these corroded and seized to the beams. Movement of the deck caused the front of the cill stones to break away. In 1976 the steel bearing plates were replaced by laminated rubber bearings set on new precast concrete bed stones. (Fig 2a and Fig 2b)

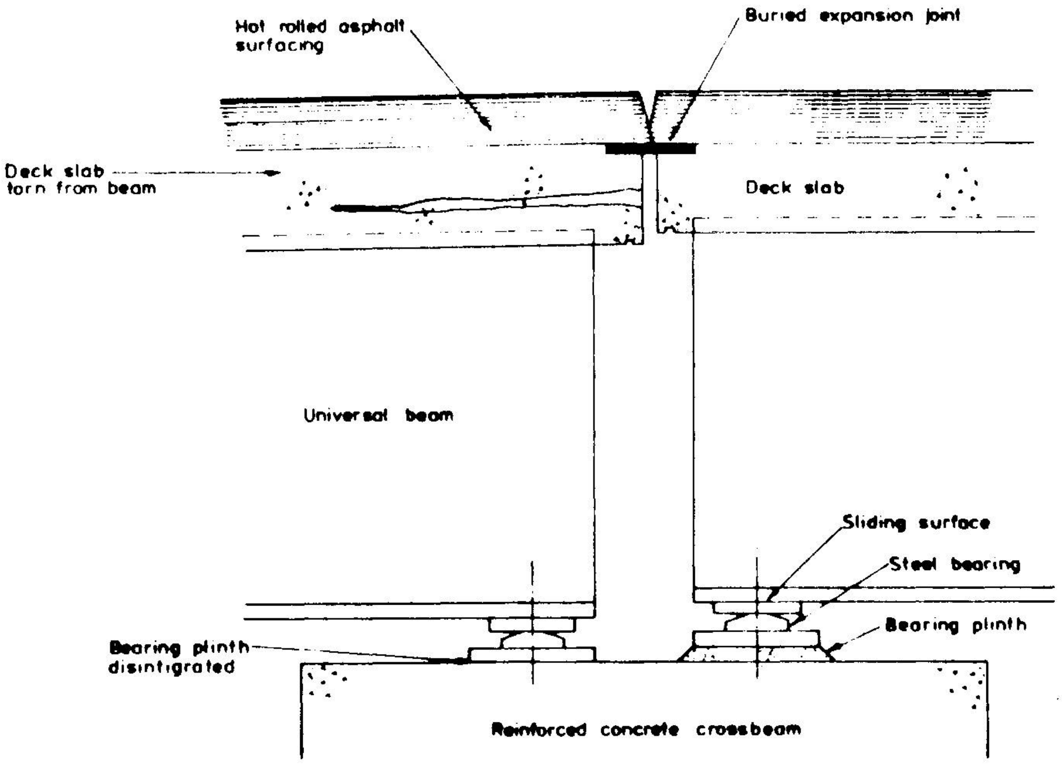

Fig. 1a Midland Links Viaduct - Detail of Bridge Bearings

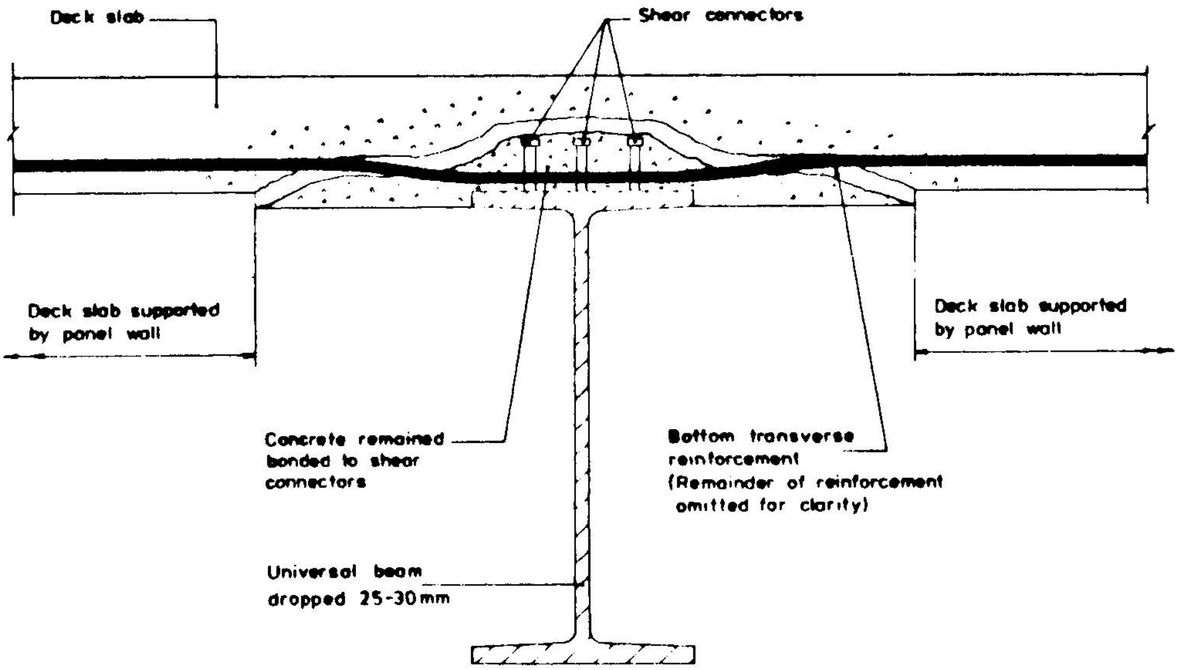

Fig. 1b Midland Links Viaduct - Typical Cracking in Deck Slab over Beams

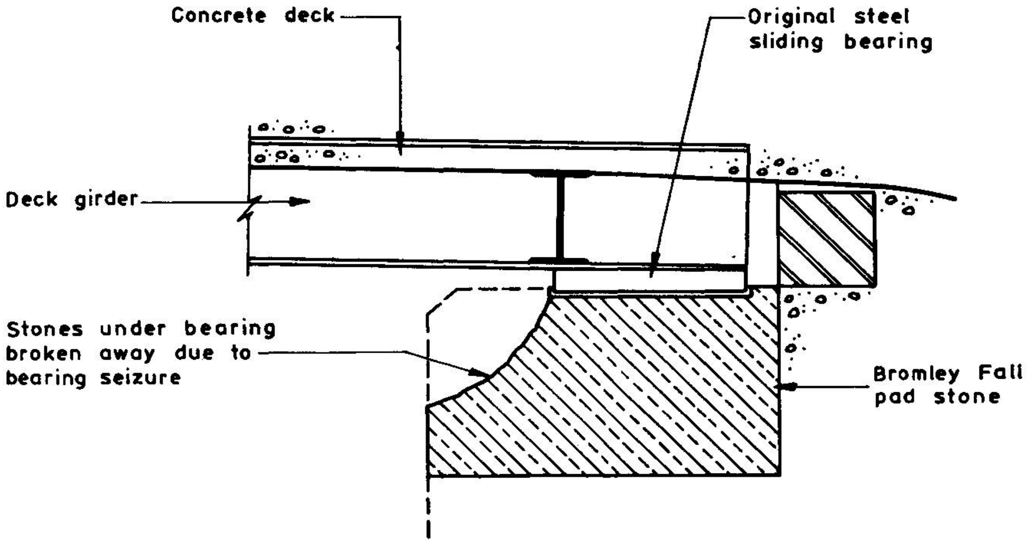

Fig. 2a Vauxhall Bridge - Original Bearing

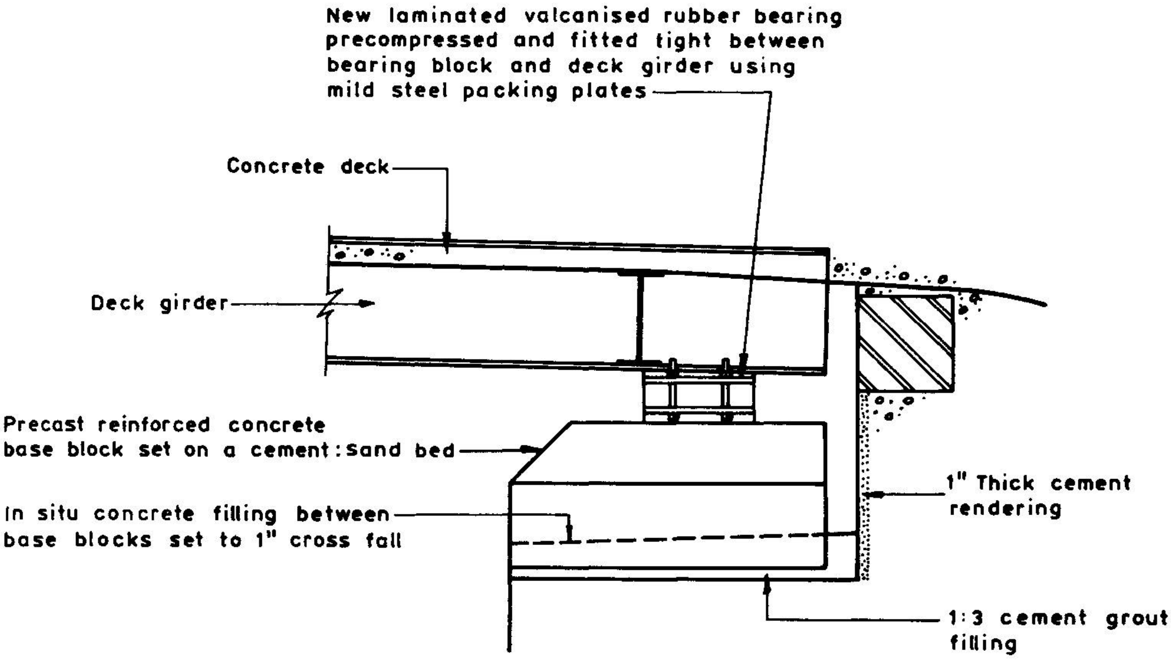

Fig. 2b Vauxhall Bridge - New Bearing in Position

3.13

The abutment bearings of Wandsworth Bridge over the River Thames in London consisted of large knuckle leaf bearings supported on a bank of four flat sided forged steel rollers tied together with side bars bolted to each roller. The rollers ran on a bottom casting. As the bearings were subject to uplift, the lower casting of the leaf bearing was tied down to the bottom casting by four 1½" diameter bolts which passed through slotted holes in the middle, or lower leaf bearing, casting. The bottom casting in turn was bolted down to the concrete abutment bearing shelf. The bridge was built in the late thirties and inspection of the bearings in 1973 indicated that although the main castings and forged steel knuckle pins were in good condition, the forged steel rollers were badly corroded with no sign of any lubrication having been applied or protection against the entry of dirt or moisture. Several of the side bars had come adrift due to corrosion of the fixing bolts and a number of the tie down bolts had broken or bent due to the heads binding on the intermediate casting. The bearings have subsequently been replaced by steel rocker bearings incorporating a PTFE/stainless steel sliding element. These have been set on new bearing plinths. (Fig 3a and Fig 3b) No provision had been made for an expansion joint in the deck surfacing, which consequently cracked at the abutment, allowing water to penetrate down to the bearings.

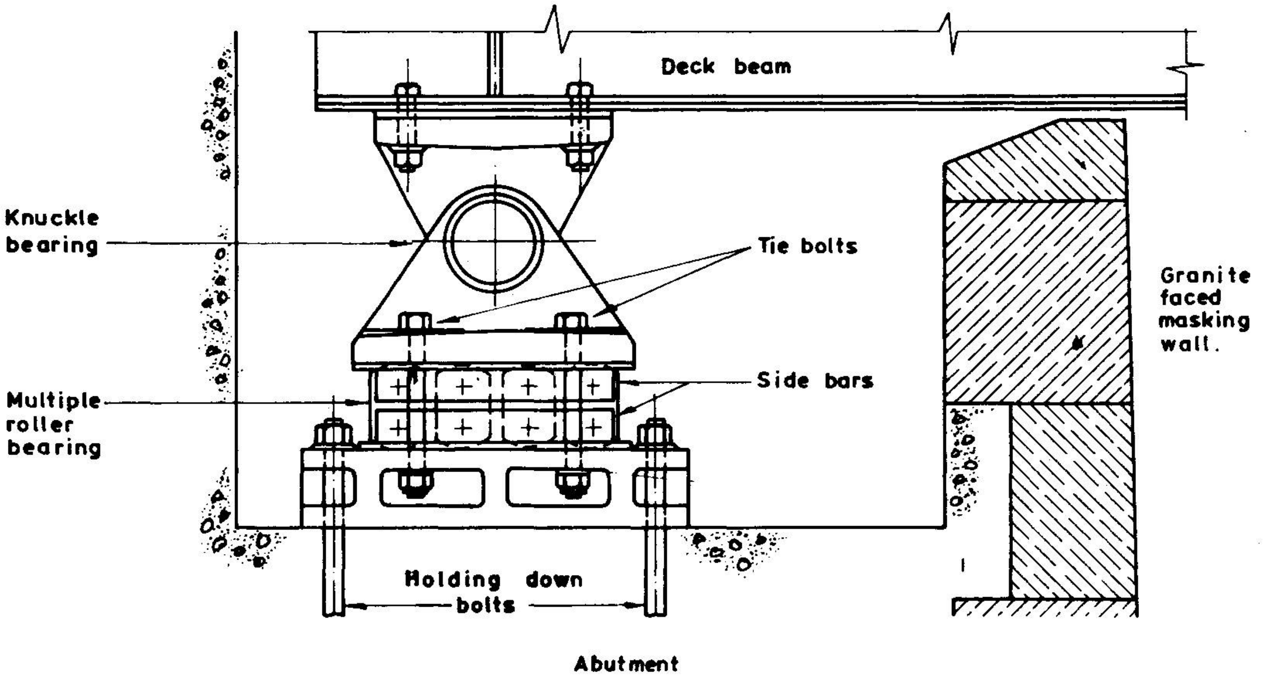

Fig. 3a Wandsworth Bridge - Original Bearing

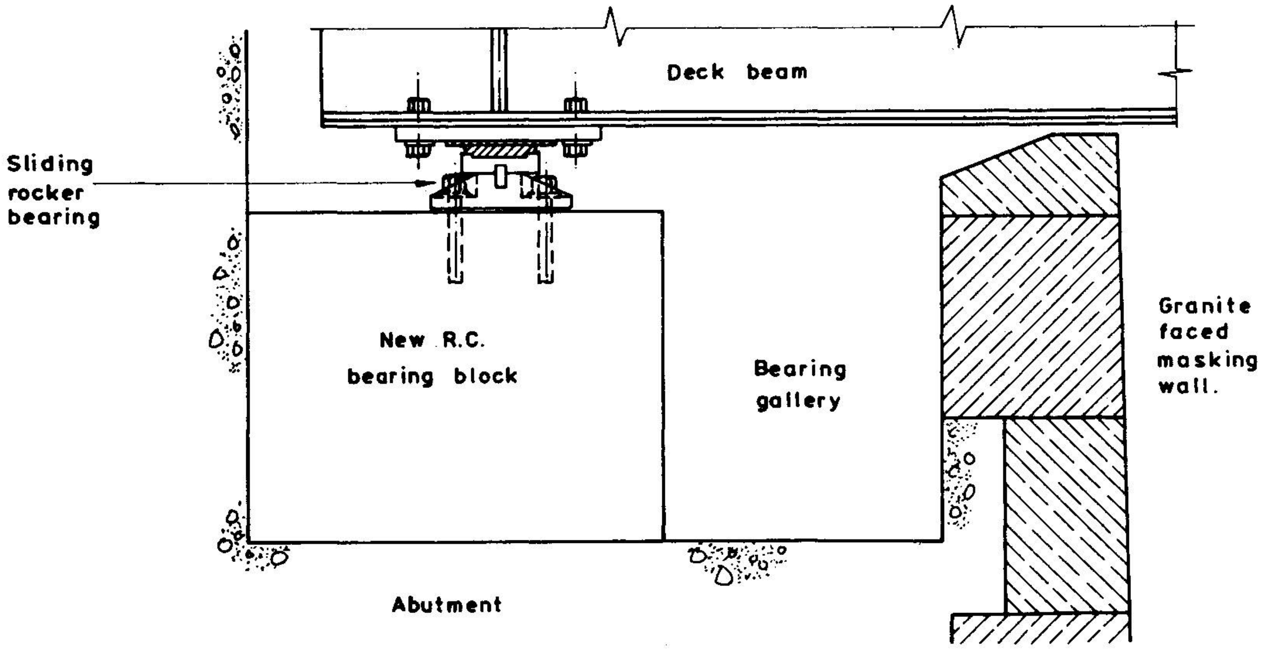

Fig. 3b Wandsworth Bridge - New Bearing in Position

3.14

In the case of bearings carrying the steel portal frames supporting part of the elevated motorway known as Westway in London, England, two problems became apparent in use. The bearings consist of a lower casting housing a block of rubber which supports an upper piston casting. Let into the upper surface of the piston is a sheet or proprietary material known as DU which consists of a 1/16 inch thick bronze plate onto which is sintered a bronze/PTFE matrix. The DU material is bedded on an 1/8 inch thick layer of asbestos based compressible material to take up any local high spots. Overlaying the DU material and forming a sliding surface with it is a stainless steel sheet fixed to a top casting. In order to prevent dirt and dust getting between the sliding surfaces, scraper bars are provided to wipe the stainless steel sheet in each direction of travel and the bearings are enclosed by removable side and end plates.

3.15

Some time after installation it was observed that the stainless steel had buckled on certain bearings. It was found that on these and other bearings the stainless steel sheet was binding on the unmachined shoulder of the piston at each end of the DU material. This was due to the elastic deformation of the portal leg base, which was formed from 1 inch thick steel plate, stiffened internally and resting on the 3 inch thick top casting of the bearing. Although only of the order of 0.03 inch at the extreme edge, this was sufficient to take up all the working clearance which had deliberately been kept to a minimum to reduce the possiblities of dust contamination of the sliding surfaces. The problem was rectified by inserting additional stainless steel/DU sandwiches and forming a slot in the shoulders by the "Metalock" method and wedging down the material above to provide additional working clearance. (Fig 4)

3.16

On some bearings excessive friction of up to 20% was recorded compared to the design value of 5%. This was found on examination of a dismantled bearing to be caused by the presence of particles of ferrous and cementitious materials normally associated with a construction site. This problem was also rectified by the use of new stainless steel/DU sandwiches. All the other Westway bearings, which include elastomeric, pot and biaxial curved sliding types have performed without trouble for over ten years.

3.17

Surprisingly, urban elevated roads seem to be prone to damage by fire, often caused by vandals setting alight flammable material stored underneath. As far as our experience is concerned no bearings have been harmed following fires although a number of decks have had to be repaired. But there must always be a first time.

3.18

Other problems that have come to light include roller bearings which have overrun their design travel so that the gear pinions ran off the end of the guidance rack and were sheared off when trying to re-engage on their return; end flanges sheared off rollers due to insufficient allowance for side thrust on these bearings. Compatibility of steelwork fabrication with the drawings is necessary if the bearings are to function in accordance with the design.

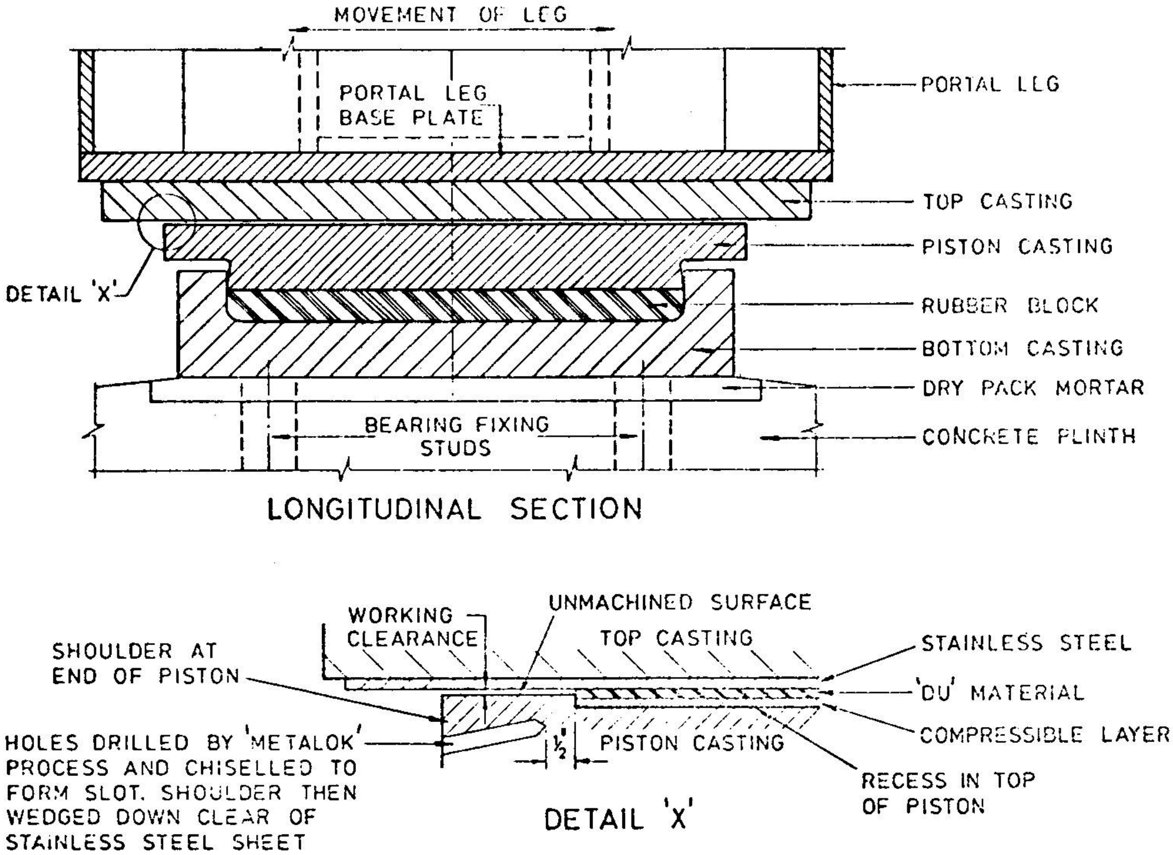

Fig. 4 Westway Section 6 - Details of Bearings

3.19

Replacing bearings can be a very difficult operation unless suitable provision has been made in the design of the bridge structure for proper access to the bearings and jacking of the bridge deck to be undertaken. Long [8] has dealt with the problems of replacing bridge bearings.

3.20

Elastomeric bearings should not be subjected to tension stresses at any time. One problem in this respect is the initial rotation due to the hog or precamber of precast prestressed beams when these are first landed on the bearings unless they are temporarily supported until the bedding mortar hardens, which can be an expensive operation. A recent innovation to overcome this problem is Andre Load Plugs (ALPS) which is currently the subject of a patent application.

3.21

ALPS are simple elastomeric plugs, which fit into holes in the body of the main bearing and stand proud in order to carry the initial loading of structural precast and prestressed beams. (Fig 5) The plugs are of a rubber compound formulated within tight limits which give the right combination of flexibility and compressive strength to ensure that the beam is supported clear of the bearing at any angle of rotation likely to be encountered during installation.

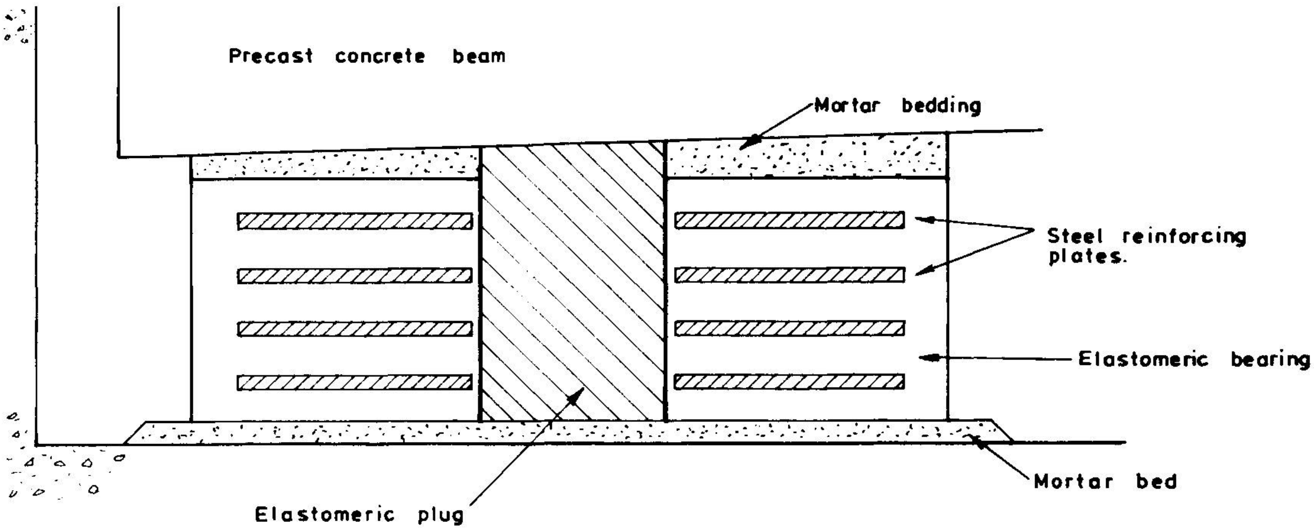

Fig. 5 Andre Load Plug

3.22

Once the beam has been lowered into place, the elastomeric plugs will carry its weight and accommodate any tendency to hog or sag. In this way, the engineer can ensure that the bedding mortar distributes the load evenly between the soffit of the beam and the top surface of the bearing. ALPS are an integral component of the bearing and remain in position when the full loading is applied. It will be interesting to gain experience of their performance in the future.

4. DEVELOPMENT OF EXPANSION JOINTS

4.1

It is important to appreciate that expansion joints are located in the most vulnerable position possible on any bridge, situated at surface level where they are subject to impact and vibration of traffic and exposed not only to the effects of natural elements such as water, dust, grit, ultra-violet rays and ozone but also the effects of man applied chemicals such as salt solutions, cement alkalis and petroleum derivatives.

4.2

To function properly, bridge expansion joints must satify the following conditions:

- accommodate all movements of the structure, both horizontal and vertical;
- withstand all applied loadings;
- have a good riding quality without causing inconvenience to any class of road user (e.g. cyclist, pedestrian);
- not present a skid hazard;
- be silent and vibration free in operation;
- resist corrosion and withstand attack from grit and chemicals;
- require little or no maintenance;
- allow easy inspection, maintenance and repair.

Penetration of water, silt and grit must be effectively prevented or provision made for their removal.

4.3

Advice on the selection, design and installation of expansion joints is given in a number of publications [1, 9, 10] . Selection of joint type is largely determined by the total range of movement to be accommodated. In multi-span viaducts one large joint is preferable to a number of small ones unless the span arrangement is such as to permit continuous surfacing over the joints.

4.4

With structures curved in plan or with skew joints, the relative movement may not be normal to the line of the joints. (Fig 6a) This can lead to binding of the elements or high shear forces in filler materials which may be exuded and carried off by the traffic, so causing ultimate failure. It is therefore important to assess this transverse displacement and to design the joint accordingly or eliminate the movement by restraining the bridge either at the joint or preferably at the bearings.

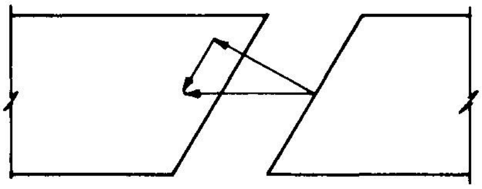

Fig. 6a Normal and Shear Displacements across Skew Joint

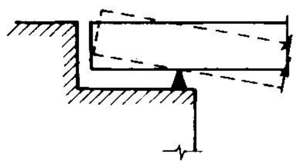

Fig. 6b Discontinuity of Joint due to End Rotation

4.5

A similar type of problem may be experienced through excessive flexural rotation at the joint (fig 6b) but, by arranging the end bearing and expansion joint in the same vertical plane, the discontinuity in the vertical direction can be minimized and the motion reduced to a purely horizontal displacement. On joints designed for small movements, this may also be overcome by providing an articulated running-on slab.

4.6

For movements of less than 5mm (0.2in), it is usually considered that no special provision is necessary, and for movements up to about 20mm (0.8in), the most popular treatment is for a gap-filled joint with continuous surfacing. (Fig 7) This can be a very satisfactory joint if carefully formed; the filling is protected by the surfacing which also absorbs much of the impact. The joint itself will not be waterproof, so it is always expedient to provide drainage under the joint to avoid staining on abutments and columns.

4.7

For movements up to about 50mm (2.0in) the most popular type of joint is the preformed flexible sealing strip compressed between nosings. (Fig 8) These joints are particularly suitable where pedestrian, cycle or animal traffic has to be accommodated as they provide a continuous surface. However, the engineer should satisfy himself that arrangements for accommodating kerbs, edge beams and medians are adequate as it is often at these points that trouble starts.

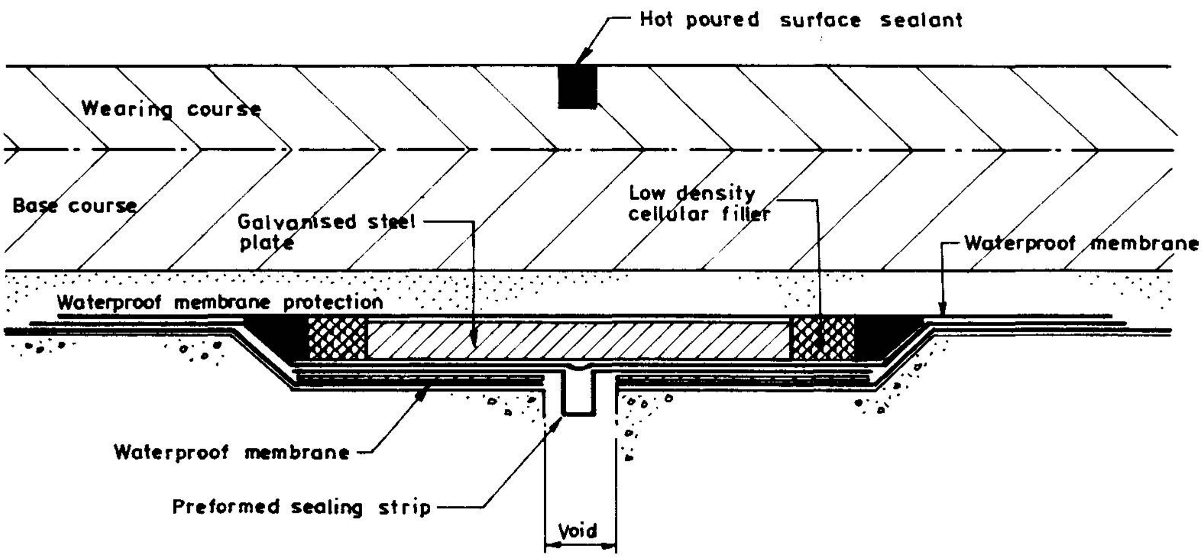

Fig. 7 Buried Deck Expansion Joint for Movements ± 10

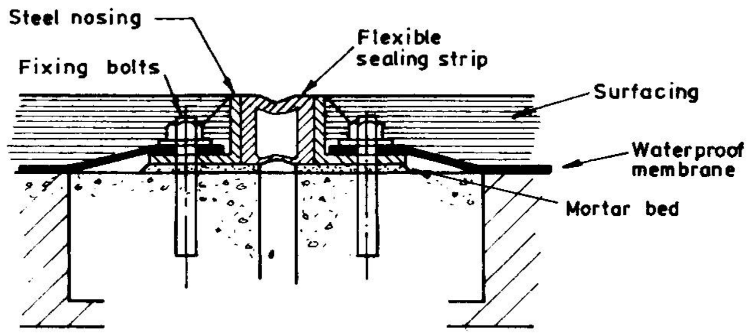

Fig. 8 Typical Expansion Joint for 20 - 50 mm Movement

4.8

Most of the cellular fillers are based on rubber or neoprene, although foamed and expanded plastics are used. Neither solid rubber or neoprene nor expanded neoprene are now recommended, as the former are expensive in relation to the small permissible compressive strains and the latter appears to suffer loss of elasticity at low temperature and with time.

4.9

Although the normal wearing properties are good, they are subjected to severe treatment and prone to damage; provision for easy maintenance is therefore essential. It is the author's opinion that these joints should be conservatively regarded as gritproof rather than waterproof. It is therefore prudent to provide at least elementary drainage on the underside and to arrange surface slopes and gully positions so as to prevent as much water as possible from reaching the joint.

4.10

During the early 70's epoxy mortar nosings were popular due to their relative cheapness and ease of maintenance but these have not always stood up well in service, the deterioration usually being attributable to poor workmanship at installation. Very many epoxy nosings have been replaced. One material favoured for this purpose is Monojoint HAC, a cement based nosing material incorporating wire fibre reinforcement. This effectively eliminates the two major factors thought to contribute to the failure of epoxy nosings; i.e. the differing coefficients of expansion of concrete and epoxy mortar causing shear forces on the bond plane and the exothermic action of epoxy mortar under cure producing shrinkage stresses. Another material used for replacing epoxy nosing joints and repairing damaged buried joints is Therma-joint. This consists of a combination of single sized roadstone aggregate and a specially formulated rubberised bitumen compound. Reports indicate that this material is standing up well under traffic conditions.

4.11

If steel sections are used to form the nosings they should be of robust construction. Many joints have been proved unsatisfactory because of failure of the fixings and, where angles are used to reinforce the opposing edges of the structure, care should be taken to achieve adequate compaction of the concrete under the angles. It is also advisable to provide suitable protective treatment to the holding-down arrangements in this region. Reinforcement used as the cast-in anchorage should be attached to the plates with full-strength welds.

4.12

A variant of the joint incorporating a preformed seal comprises a flexible gland or strip of reinforced neoprene set in nosing blocks of solid neoprene, reinforced with steel plate, which are bolted down to the bridge deck and abutment structures.

4.13

An alternative to the cellular preformed sealer consists of a shaped slab of neoprene reinforced with steel. (Fig 9). Movement is accommodated by shearing strains in the elastomeric material which is specially shaped and often reinforced with steel plates to enable the joint to span the expansion gap without deflecting significantly under load. The joint is bolted down flush with the wearing surface and, as it can be manufactured in one continuous piece to any desired length, it is waterproof, although care must be taken with the details at kerbs and edges. To reduce the hazard of skidding prevalent with wet rubber, some anti-skid treatment, usually grooving of the top surface, is applied.

4.14

Joints for large movements are usually of the open type using sliding plates, cantilever or propped cantilever tooth or comb blocks. (Fig 10) Modular compression sliding systems have been developed over the last decade. It is seldom practical to seal joints where the total movement exceeds 50mm (2in) and adequate provision must therefore be made for the disposal of surface water, grit, salt, etc., with easy access for maintenance. Because of the passage of surface waters through the joint and splashing in and around the collector system, it is vital that adequate protective treatment should be applied to any parts of the joint exposed to these corrosive elements.

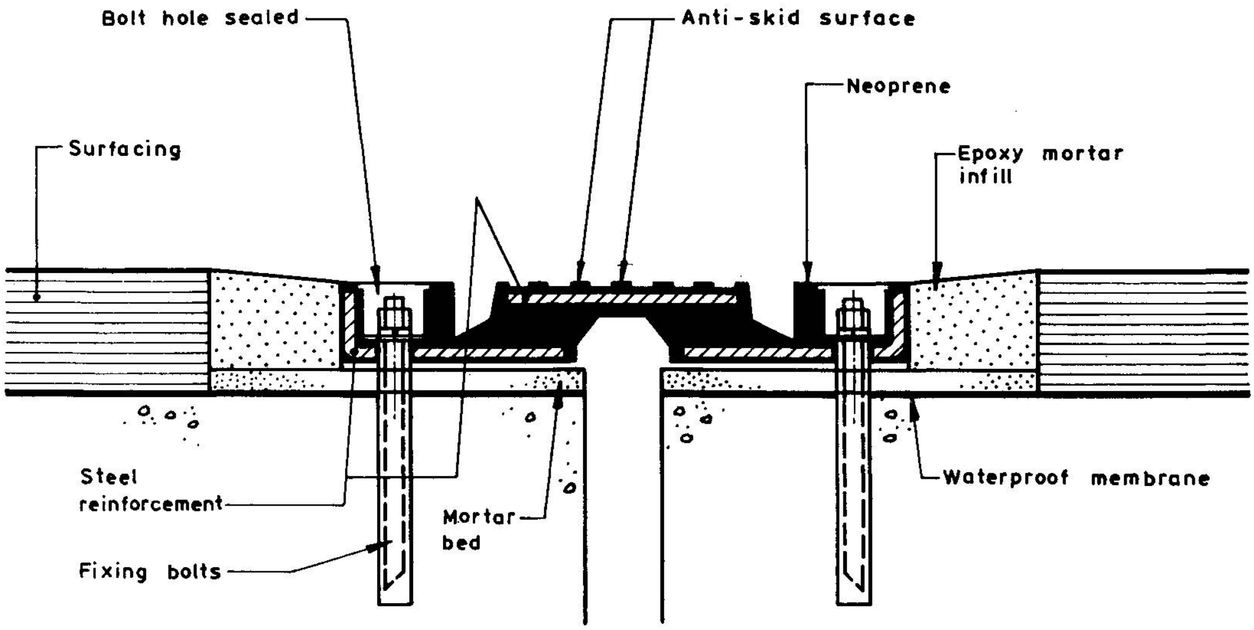

Fig. 9 Cushion Joint

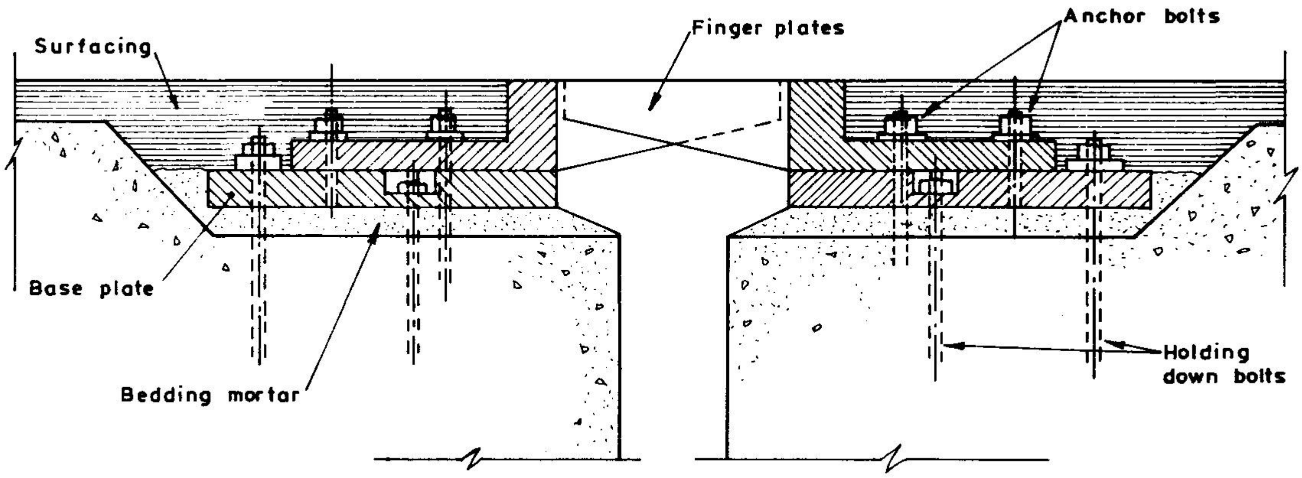

Fig. 10 Cantilever Joint

4.15

For movements up to about 130mm (5in), toothed plates can be cut from a plate 40mm ($1\frac{1}{2}$") thick but for larger movements it is preferable for the comb blocks, having long narrow teeth tapering in depth to suit the applied moment to be cast or fabricated by welding. Castings may be made from either cast steel or spheroidal cast iron: the latter is preferred as it gives less trouble in the casting process.

4.16

On motorways where cyclists and pedestrians are excluded, a smoother ride can be obtained without cover plates but on all purpose roads covers are necessary on safety grounds to reduce the gaps between the teeth. If cover plates are required on comb joints, they should be securely fixed and preferably welded to the teeth at frequent intervals to prevent chatter. Because of chatter, sliding plate joints are not popular for carriageways but they do offer a cheap solution to footway joints if care is taken to keep them free from grit. The CIPEC joint for movements up to 160mm (6in), incorporates an elastomeric compression sealing element below the teeth which are triangular in plan to allow for shear displacements.

4.17

Modular expansion joints consist of a series of compression seals between shaped metal beams running the length of the joint. The longitudinal separation beams are supported on short cross beams spanning the joints. Provision has to be made for the cross beams to slide on their supports during expansion and contraction of the joint. Problems related to modular compression sealing systems have been expounded by Watson [11]. These include buckling, bending and tilting of the separation beams, objectionable noise and leaking. These problems are overcome in the Maurer joint by welding the separation beams to individual support beams which, in turn, are held under pressure on glass fibre reinforced PTFE resilient bearings. (Fig 11) The seals are mechanically locked into the separation beams and do not rely on compression or adhesion for maintaining watertightness. The Maurer joint is manufactured to high dimensional standards. The adjacent concreting and fixing of reinforcement has to be of similar standards if the joint is to fit properly.

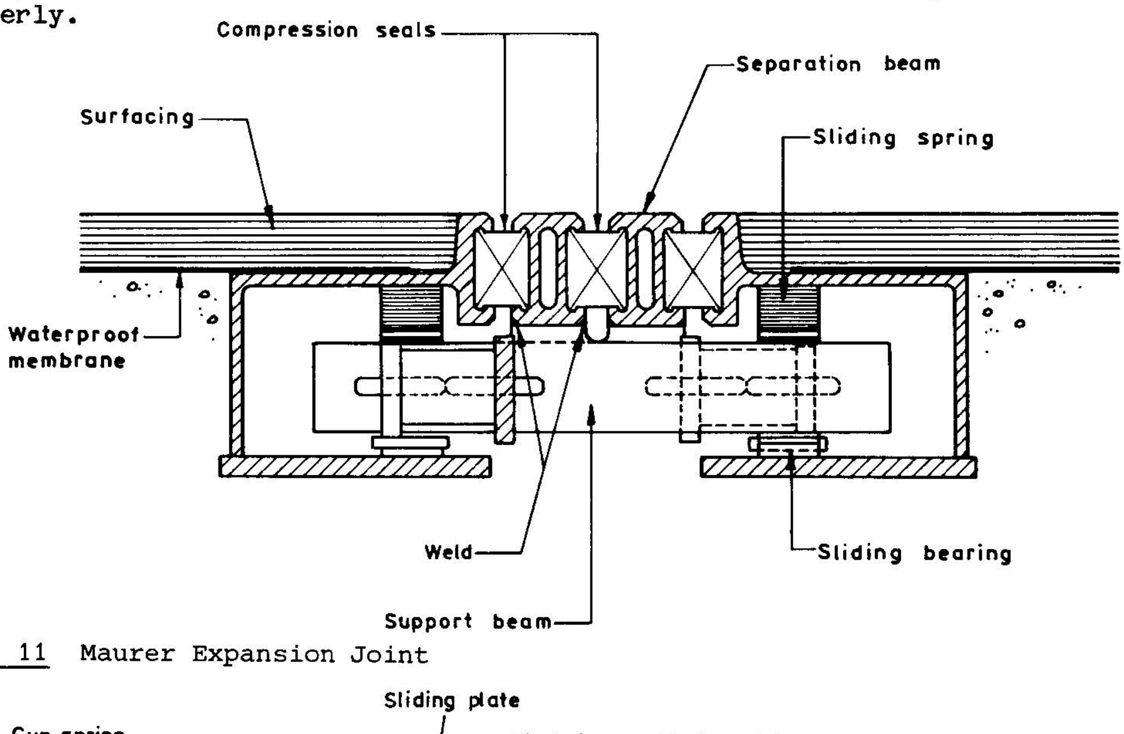

Fig. 11 Maurer Expansion Joint

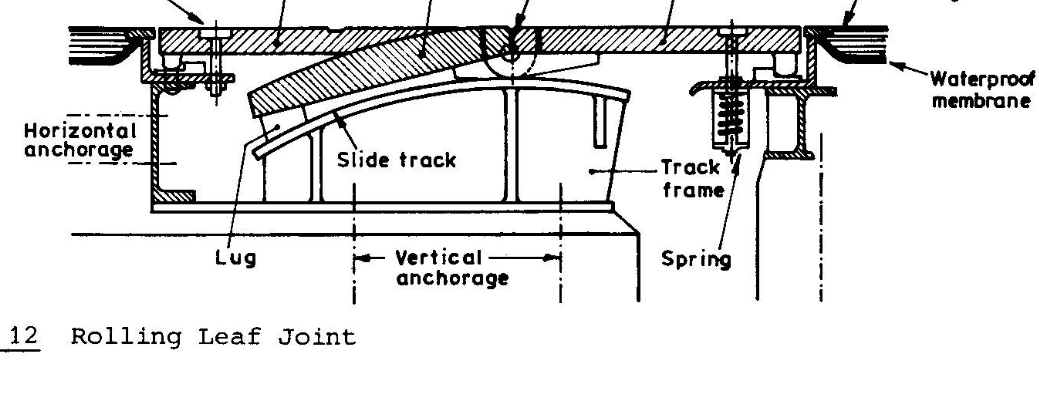

Fig. 12 Rolling Leaf Joint

4.18

For very large movements, rolling leaf or articulated plate type joints, as produced by Demag AG, are recommended. (Fig 12) These joints are robust and have a good service record. Where such joints or any joint which presents a large area of metal as the running surface are used, the skid risk can be high but can be effectively reduced by a coating of calcined bauxite in an epoxy resin mix.

5. ACKNOWLEDGEMENTS

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The document, which has been issued for public comment, was prepared by a sub-committee of the British Standards Institution under the chairmanship of Dr. O.A. Kerensky, FRS, FEng, and issued under the authority of the BSI Technical Committee for Steel, Concrete and Composite Bridges.

Greater London Council have graciously permitted publication of details of Structures under their jurisdiction.

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Economic Aspects in Planning of Bridge Rehabilitation and Repair

Aspects économiques et planification de la réparation des ponts

Wirtschaftliche Aspekte der Planung für die Sanierung und Instandstellung von Brücken

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SUMMARY
The cost for the overall upkeep of bridges on the Danish National Highway System is expected to increase sharply in the coming decades. The paper endeavours to predict this increase and suggest policy models for the management of highway bridges under conditions of temporary insufficient available public means. When needed repairs and rehabilitations must be postponed due to lack of funds it becomes vital to make the best out of the available means as well as to keep the increase in risks under control. The development of advanced management information systems becomes thus a necessity.

RESUME
Le coût de l'entretien général des ponts du réseau danois des routes augmentera fortement dans les décennies à venir. Le présent document essaie de prédire cette croissance et propose des modèles de décision pour l'administration des ponts, tenant compte des limitations de crédit. Lorsque les réparations nécessaires sont reportées à plus tard, par suite de moyens financiers insuffisants, il devient indispensable d'utiliser les fonds disponibles au mieux et de contrôler les risques croissants. Un système évolué de gestion de l'information devient alors indispensable.

ZUSAMMENFASSUNG
Es wird angenommen, dass die Kosten für den allgemeinen Unterhalt von Brücken im dänischen Autobahnsystem in den kommenden Jahrzehnten stark ansteigen werden. Der vorliegende Bericht beruht auf einer Wachstumsvorhersage und schlägt Verfahrensmodelle für die Leitung von Autobahnbrücken unter der Bedingung vorübergehend ungenügender Finanzmittel vor. Wenn notwendige Sanierungen und Instandstellungen aufgrund nicht vorhandener finanzieller Mittel verschoben werden müssen, zeigt es sich als angebracht, aus den vorhandenen Mitteln das beste zu machen und die erhöhten Risiken unter Kontrolle zu halten. Die Entwicklung eines administrativen Informationssystems ist unerlässlich.

1. BACKGROUND FOR THE PRESENT BRIDGE SITUATION

In the last decades the Danish Highway Directorate has mainly been engaged with the construction of the Danish motorway system including a large number of bridges. These vast construction activities have been carried out at times under hectic circumstances with the aid of a major influx of personnel at all levels with limited experience in bridge building. The optimistic atmosphere of the previous decades fostered many new ideas, that were often adopted uncritically although not all of the accepted changes were of equal value.

Under the pressure of a strong public demand to establish the needed new road systems here and now, the governing idea during this construction period were to construct as much as possible with as little as possible without seriously considering the long term effects of such an approach. To-day the situation is revised. Public opinion has a very short memory. What yesterday was most desirous is to-day a disgusting intrusion upon a new set of environmental values.

In this adverse political climate we are facing the long term effects of previous years of construction booms with problems of unexpected early decays of modern bridge structures. Concrete structures and especially prestressed concrete structures pose the more serious technical challenges with demands of a thorough structural understanding combined with an extensive knowledge of materials technology especially of concrete, it´s deteriorating processes and the adverse conditions towards the overall durability of the bridges.

The economic consequences of the present situation is not fully appreciated or understood, but it is obvious that an increase of expenditures in the overall upkeep of bridges is to be expected.

It is, however, not quite so obvious that the expected increase of the costs will be met with the same degree of understanding by appropriation authorities as was the case during the hey-days of road construction of the sixties and seventies.

2. A PREDICTION OF FUTURE BRIDGE COSTS

2.1 Some Basic Economic Definitions.

Before making a prediction of costs it is necessary to clarify what kind of activities are covered by the various cost elements. The following definition has been made to serve the aim of the subsequent prediction and does not reflect any actual accounting system.

Future costs of bridges - excluding proper new constructions - are in the following subdivided into three elements:

- The costs to routine operations and maintenance.
  We deal here with the daily operations and the costs of them, that - apart from a suitable organization and a thorough planning - mainly is a function of the size of the total bridge volume and only to a lesser degree is influenced by the average age of the bridge volume.

- The costs of repairs.
  In this connection repairs are defined as the mending of existing structural elements such as surface treatment as painting or repair works on concrete but excluding all kinds of replacements.

- The costs of replacements.
  These costs can cover replacements of bridges as well as parts hereof. Some bridge elements are replaced relatively frequent such as bridge surfacing and waterproofing, parapets, joints and railings. Other elements are replaced considerably less frequent such as bridge decks, retaining walls, columns and foundations.

2.2 The Costs of Replacements.

The replacement costs of a bridge or a bridge element include the total costs of the replacements, i.e. the demolition plus the reconstruction plus arrangements for traffic deviation during the reconstruction but excluding all additional costs imposed upon the road users during the replacement of the bridge or bridge element.

The replacement costs will first appear when the lifespan of the bridge or the bridge element is terminated, which will happen either when the bridge or the bridge element is worn out or deteriorated to a degree that makes repairs economically unsound, or if the bridge has become functionally obsolete.

If we disregard the case of obsolete ability to function, then there is a close relationship between the total costs of repairs in a bridge's lifetime and the time when a replacement is needed. Economically, repairs aim to postpone replacements until the time when a replacement is more profitable. It is therefore often technically possible - but hardly economically feasible - to avoid a threatening replacement by extensive repair works.

2.3 The Life Expectancy of Bridges

The following assumptions cover, strictly speeking, only bridges on the Danish national highway system. The assumptions are, however, supported by similar expectations for bridges on highway systems, e.g. in West Germany, Sweden and some parts of The United States.

As mentioned above various bridge elements deteriorate with different rates. We replace bridge surfacing and waterproofing considerably more frequent than structural elements in the superstructure. Parapets, joints, railings and other bridge elements have a relative short span of life. In the following the bridge volume is divided into 20% of the total replacement costs (corresponding to bridge surfacing, waterproofing, parapets, joints, railings etc.) with an average life expectancy of 20 years with a possible minimum of 5 years and possible maximum of 35 years. 20% of the total bridge volume has by these assumptions a mean life expectancy of 20 years with a standard deviation of 6 years. The remaining 80% of bridge volume is assumed to have an expected length of life of about 60 years with a possible minimum of 20 years and a possible maximum of 120 years. 80% of the total bridge volume has thus a mean life expectancy of 64 years and with a standard deviation of 20 years.

The above assumptions are indeed very rough and possibly too much so, but the assumptions are maintained for reasons of simplicity and clarity as well as of lack of a more informed basis of conjecture.

For those who find the assessed life expectancies surprisingly low, it should be noted that the climate in Denmark corresponds to an average winter period of 4-5 months with many often daily temperature changes around freezing point with high humidity, rain, snow and ice. These climatic conditions deteriorate especially concrete structures very rapidly and in addition to the climatic effects a large number of concrete bridges suffer from alkali-silica reactions that together with the use of de-icing salt acts as a "catalyst" in the deterioration processes.

ASSUMED LIFE EXPECTANCY OF ROAD BRIDGES

Components	Share	Life expectancy
Pavement Waterproofing Rails Bearings Joints etc.	20 % of the total replacement costs	Expected average = 20 years Expected minimum = 5 years Expected maximum = 35 years **Mean life expectancy = 20 years** **Standard deviation = 6 years**
Bridge decks Columns Foundations Retaining Walls etc.	80 % of the total replacement costs	Expected average = 60 years Expected minimum = 20 years Expected maximum = 120 years **Mean life expectancy = 64 years** **Standard deviation = 20 years**

Figure 1

With a mean life expectancy for bridges of 64 years (half of the bridges are thus expected to be between 64 and 120 years) it is at the same time assumed that the functional demands upon the bridges, such as the load carrying capacity, the width and the clearence will not need to be changed significantly in a corresponding long period. This assumption should be compared with the fact that in Denmark the allowable high of vehicles has been changed from 3.6 mtrs to 4.0 mtrs within the last 5 years and that the codified traffic load (for exceptionally heavy vehicles) has been doubled within the last 20 years.

2.4 Future Bridge Costs on the Danish National Highway System

Figure 2 shows the total bridge volume on the Danish national highway system distributed according to the decades in which the bridges were built or is planned to be build, and measured not in the yearly construction costs but in yearly added replacement costs. The profile of the total bridge volume corresponds with similar profiles of other national highway networks (give and take a decade) and shows the relatively high construction intensity during the depression before the second world war, almost no construction during the war and the decade following the war, where upon the construction of bridges increased strongly during the building of national motorways that in the case of Denmark is expected to be completed inside the present decade.

BRIDGES ON THE DANISH NATIONAL HIGHWAY SYSTEM

Yearly investments measured as replacement costs
in mio. of Danish kroner

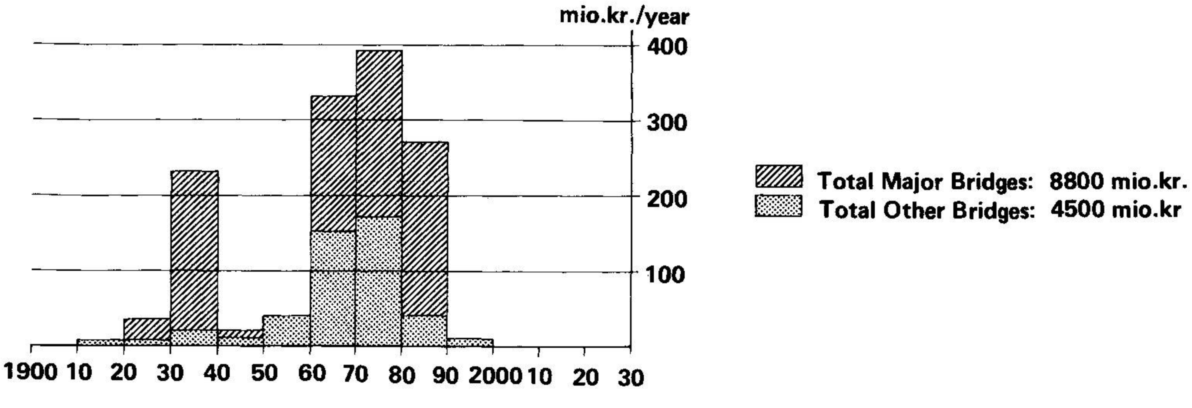

Figure 2.

These activities cause the above mentioned costs to operation, maintenance, repairs and replacements. Do we project the above mentioned assumptions regarding the costs of replacements into the shown activity of construction then we get the future yearly costs of replacements as shown in figure 3. As expected we are facing sharply increasing costs for replacements and almost half of these costs will be commanded by the relatively short-lived 20% of the bridge volume consisting of surfacing, waterproofing, railings, joints, bearings, etc.

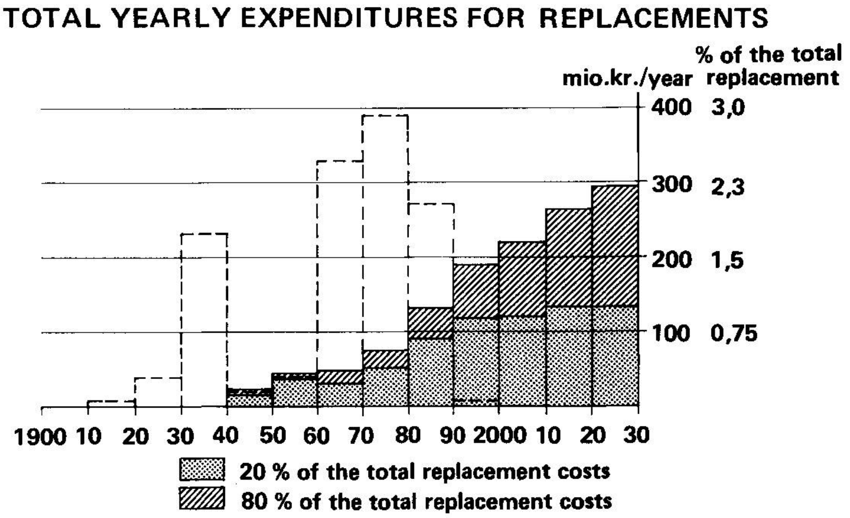

Figure 3.

To the replacement costs we must add the costs to operation, maintenance and repairs. The costs to operation and maintenance, which relate to daily activities, is regarded as practically independent of the age of the bridges. It is assumed that the yearly costs hereto is about 0.5% of the replacement costs of the actual bridge volume. The costs to repairs are of cause dependent on the age of the bridges. It is assumed that the average costs to repairs are of the same order as the costs to operation and maintenance, i.e. about 0.5% of the total replacement costs of the actual bridge volume but these costs vary from zero to about 1% during the life time of the bridge. With these assumptions we get a cost profile as shown in figure 4 for operation, maintenance and repairs.

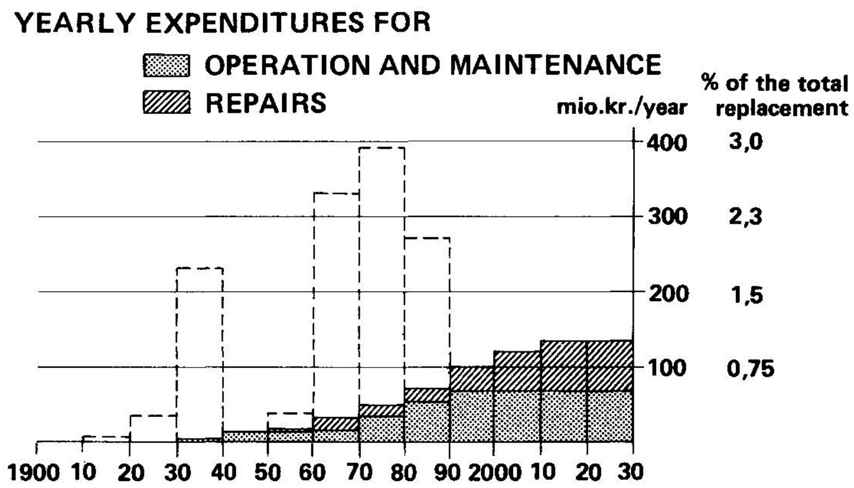

Figure 4.

When all the costs add together we get the fearsome picture as shown in figure 5. It is seen that in the eighties we must use almost twice as much as in the seventies, and in the nineties almost 2.5 times as much as in the seventies as yearly costs to the overall upkeep of the bridges. If we move 50 years ahead of present time we will, with the assumptions made, reach a level of about 3% of the total replacement costs as yearly expenditures for operation, maintenance, repair and renewals of the bridge volume before the cost increases start to level of.

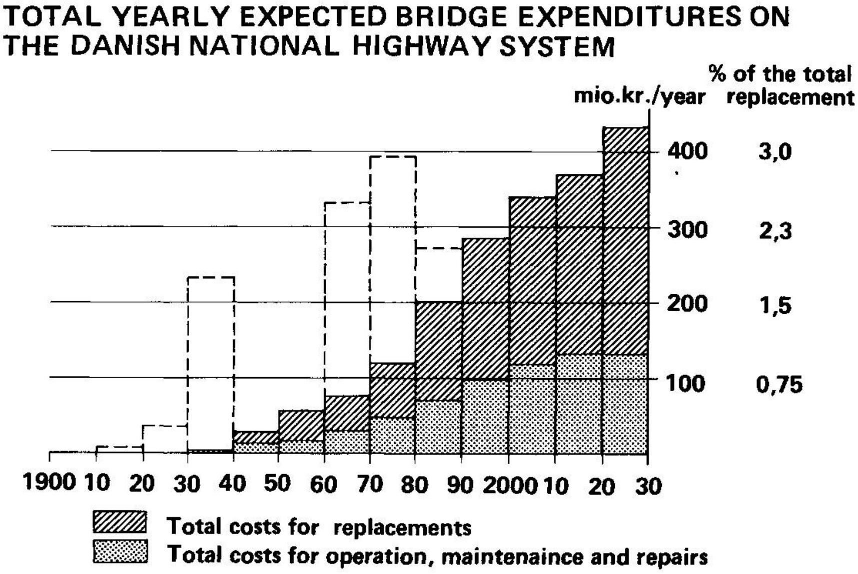

Figure 5.

Firstly, it is important to underline that the above shown prediction is a first rough attempt to make a forecast of the future costs of bridges and that considerable uncertainties are attached to the calculations which we hopefully by a more intensive planning effort can minimize and this must of cause be done. It is doubtful, however, whether such an increased planning effort will lead to considerable changes in the pattern and the order of magnitude of costs that we see here. It is on the other hand obvious that the cost increases as shown are quite unacceptable and that serious efforts are needed to limit and reduce the increases.

The fundamental problem is the durability of the bridge structures or rather the lack hereof. As shown in figure 5 it is especially the renewals, the replacements, that will draw heavyly upon our limited resources and to a large degree this is caused by the limited durability of the concrete structures. The bridges that have been built can only with difficulty been given a higher degree of durability, but the bridges that are going to build, whether it be replacements or actual new constructions, must be constructed with a fundamentally better durability. This demands a collective effort on all levels not in the least within research and development, and the argument to increase resources to such an effort lies evidently in the above shown increases in bridge expenditures which otherwise are to be expected.

3. COUNTERACTIVE MEASURES AGAINST INCREASE IN BRIDGE EXPENDITURES

Ultimately only increased durability of the bridge structures as a whole can make an impact upon the increase in expenditures. It is, however, important not to focus on the durability problems alone. Sudden increases in the functional demands such as demands for increased load carrying capacity, increased road width or clearance can make efforts in the field of durability meaningless, if the bridges have not been constructed with reasonable foresight with regard to future traffic demands. In this context it should be noted that the road networks serve an increasing part of the overall need for transport and that alternative transport possibilities from railways and shipping, when exceptionally heavy indivisible loads must be moved, become increasingly more questionable, because industry often disregard these forms of transport in their overall planning, where factors such as easily accessible, well educated labour, various environmental regulations, possibilities of expansions etc., overrules the diminishing advantages of access to railway and shipping facilities.

Even the most optimistic highway administration cannot expect immediate positive response from appropriation authorities in meeting the increasing bridge expenditures and bridge authorities may therefore - hopefully only temporarily - operate with insufficient funds.

This situation raise two principal questions:

- What are the consequences of the limited appropriations?
- How can we counteract or control such consequences?

3.1 Consequences by Limited Funding

It is possible for quite a long time to stave off the need for bridge maintenance without creating any serious problems for the traffic or road users. The fact that there is no immediate response when appropriations for bridge expenditures are cut can tempt political authorities in their some times desperate pursue for cut-backs in the public household.

The New York State Department of Transportation made a report in 1980 that most vividly described the consequences of limited appropriations in it's report "The Deterioration of New York State Highways Structures". The report analyses on the basis of 5 years of bridge inspection data, the change in the rating of the bridges in the State of New York and makes a projection of what is to be expected if appropriation authorities remain as tightfisted as they are or have been. The report concludes that the current rate of decay appears to be five times the historical rate and may increase still futher. The report also project that by the year 2010 the deterioration on New York structures will be essentially completed, when 95% will be rated deficient and more than 50% will posted for reduced load.

In connection with a change in the traffic code in Denmark where the allowable total weight of vehicles was increased to 44 ts and the allowable axle load was increased to 10 ts we made an inventory of the load carrying capacity of existing road bridges. We managed to classify 6203 bridges out of a total registered number of bridges of 8234. Of the classified bridges, 4895 bridges, or 79% could carry the new traffic demands, the remaining part or 21% failed to do so in a varying degree. For instance will 10% or 628 bridges only be able to sustain an axle load of 5 ts and a total weight of vehicle of 8 ts. Only an insignificant number of the weaker bridges have been posted. Experience have shown that posting for limited allowable load does not prevent overloading so there is potential risks of bridge collapses regardless of traffic signs. Up till now no such collapses have occured which might be due the fact that the weaker bridges are situated on minor roads where heavy vehicles rarely travel. An other factor might be that the change in the traffic code does not momentarily change the size and construction of heavy vehicles, although it is expected that in a few years time this change will be completed.

3.2 Priorities under Restricted Funding.

With restricted funds evaluation of priorities becomes a key in establishing control of the consequences. The main purpose for highway authorities is obviously to maintain the road system intact for the road users, so major or more trafficked roads will evidently take precedence over minor or less trafficked roads when allocating funds for the overall upkeep. Large or major bridges are the most costly elements in a road system and they are often especially sensitive to neglect of maintenance so such structures will command a higher priority than ordinary bridges.

A tool in such a priority system would be a discounting technique as described in the 1981 report: "bridge maintenance" from a road research group under the Organisation for Economic Co-operation and Development (OECD). In this report it is suggested that a future cost, $C_{T=t}$, is expressed by its present equivalent, $C_{T=0}$:

$$C_{T=0} = e^{-r \times t} \cdot C_{T=t} \qquad \text{(continuously}$$

or $\qquad\qquad$ or

$$C_{T=0} = (1 + r)^{-n} \cdot C_{T=t} \qquad \text{discretely)}$$

where t is a given time span, n the equivalent number of time periods, and r is the time preference rate or discount rate. The term "interest rate" is here intentionally avoided as interest rate may only be a subconcept to the discount rate.

Figure 6 shows the effect on future costs of using the concept of discounting. The abscissa gives the number of years ahead of present time when a future cost shall be carried (defrayed), and the ordinate axis gives its present value in percentage of the future cost. The relationship is shown for various discounting rates 1 %, 2 %, 3 % and up to 15 %. It is assumed that we operate at the same cost level, i.e. no inflation and the discount rate does therefore not contain the rate of inflation.

By discounting the future costs of operations, maintenance, repairs and replacements it is theoretically possible to adapt limited funds to a poorly maintained bridge stock with backlogged repairs by ajusting the discount rate, i.e. increasing the rate. The reason for this is - as mentioned above - the existing possibility of staving off bridge repair needs without causing serious problems - for a period at least. By increasing the discount rate the present value of all future costs are reduced and thus the the consequences of a low level of maintenance are viewed through the wrong end of the telescope. The question of whether such a decision is recommendable or not is ultimately a political matter. There might simply not be enough funds available - perhaps only for a limited period - and the show must go on.

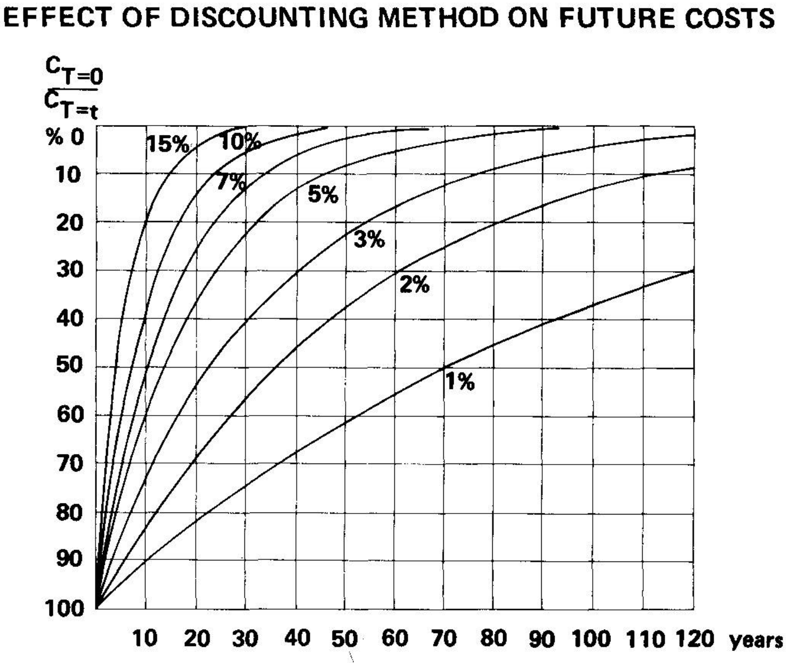

Figure 6.

In a priority system the overall discount rate is a function of available funds and accumulated needs. A low overall discount rate (1-3%) corresponds to a well maintained bridge stock with no serious backlogged repairs or replacement needs, whereas a poorly maintained bridge volume with several needs for repair and replacements and in a situation with very limited funds to meet these demands will correspond to a very high (7-15%) overall discount rate.

The overall discount rate sets the general level for the price of money. The next step is the priorities. With the overall discount rate as a "mean value" bridges on more important roads are given a low discount rate where bridges on more humble roads are given a high discount rate. A similar procedure can be introduced for major bridges against ordinary ones.

4. MANAGEMENT SYSTEMS

In the first case with sufficient funds and a well maintained bridge stock the management hereof poses few problems and only limited risks that in general do not call for more than ordinary care and diligence. The latter case, however, with a poorly maintained bridge stock, is a challenge for the best bridge management.

Information in its broadest sense is the first demand. A bridge authority does not manage inspection, maintenance, repairs, etc., but manages rather information from inspection, maintenance, repairs, etc., and the quality of the management is not likely to exceed the quality of the information received.

Bridge inspection reports from superficial, principal and special inspections form together with reports from executed repairs and the general bridge data the backbone of the information available to management. When all this is added together from just a few thousand bridges the amount of information becomes unmanageable unless steps are taken to automate the digestion of it. EDP-based databanks for general bridge data as well as from inspection reports, etc., is an obvious answer to this problem and several such databanks have been introduced. The aim for the EDP-based databanks has been easy and fast accessibility to available information and a need for analysing the large amount of collected experience from bridge inspection reports. The next generation of databanks will aspire to become sophisticated management tools containing also economic information such as repair costs defrayed against damages inspected. A further step could be the introduction of a priority system as outlined above combined with analyses of the consequences of registered damages, decays or reduced load carrying capacities with the aim of establishing a basis for management decisions of what should be done: nothing - temporary repair - full rehabilitation - and relevant solutions in between.

Such a management tool could prove to be an essentially complete risk information system where risks of collapses, deficiencies of any kind and their economic consequences as well as the level the serviceability of the road network will serve to make the most out of the available means.

Regardless of how elaborate such a risk information system may become, the increasing decay of the highway structures will continue with an increasing accumulation of backlogged repairs and replacement needs when the available funds are insufficient. The bridge information system should contain capabilities that will assist the overall planning efforts, whereby political appropriation authorities will be able to make informed decisions as to whether an accelerating decay of highway structures should be arrested by influx of funds or whether the present state of affairs is acceptable when considering the needs of competing public enteprises.

Such capabilities can be created from the data bases already established by registering the total costs of all backlogged needs combined with possibilities of regressions and forecasts of the development of these total costs under varying influx of funds into the bridge maintenance system.

In the previously mentioned report "The Deterioration of New York Highway Structures" attemps along these lines have been made. The backlogged repairs in N.Y. are estimated to amount to $ 4 billion in the year 2010 if the current rate of expenditures is maintained.

Bridge Inspection Program: a Vital Bridge Planning Program Aid

Programme d'inspection de ponts: élément essentiel de la gestion des ponts

Inspektion von Brücken: ein wichtiges Element im Brückenmanagement

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SUMMARY
The required inspection of bridges at regular intervals not to exceed 2 years provided much information on the relative condition of bridges in each governmental entity. This information can be used in the planning, budgeting and decision-making process for all phases of a bridge program from routine maintenance to the replacement of major structures.

RESUME
L'inspection requise de ponts à des intervalles réguliers, ne dépassant pas 2 ans, fournit beaucoup d'informations sur l'état relatif des ponts à chaque niveau de l'administration. Cette information peut être utilisée pour le projet, le budget et la prise de décisions dans toutes les phases d'un programme de construction, d'exploitation et de remplacement de ponts.

ZUSAMMENFASSUNG
Wichtige Informationen über den Zustand der einzelnen Brückenverwaltungen zugeordneten Brücken können aus der in zweijährigen Intervallen nötigen Brückeninspektion gewonnen werden. Diese Informationen sind von grossem Wert bei der Planung, bei Kostenvoranschlägen und im Entscheidungsprozess von Brückenunterhalt bis zum Ersatz ganzer Konstruktionen.

1. HISTORY

Until the December 1967 collapse of the Silver Bridge over the Ohio River between West Virginia and Ohio, very little support existed for a national bridge inventory and inspection program in the United States. The collapse caused 46 deaths and the public outcry and subsequent Congressional hearings resulting from the tragedy clearly supported the need for a national program. The hearings demonstrated that many States were not sure how many bridges they owned and others had no formalized inspection or recordkeeping procedures.

As a result of these hearings, the Congress, in the 1968 Federal-aid Highway Act, directed the Secretary of Transportation "in consultation with the State highway departments and interested and knowledgeable private individuals . . . to establish national bridge inspection standards . . . for the proper safety inspection of bridges on any of the Federal-aid highway system." The law required each State to maintain a current inventory of all bridges on the Federal-aid system.

In the 1970 Federal-aid Highway Act, the Congress directed the Secretary in consultation with the States to inventory all bridges on the Federal-aid highway systems over waterways and other topographical barriers, classify them according to their serviceability, safety and essentiality for public use and assign each a priority for replacement.

On April 27, 1971, the National Bridge Inspection Standards (NBIS) were issued to satisfy the mandate of the Congress. By the end of 1973, most States had inventoried all bridges on the Federal-aid highway systems.

In the 1978 Surface Transportation Assistance Act, the Congress directed the Secretary of Transportation to extend the inventory and inspection program to include all highway bridges on public roads. The inventory was to be completed by December 31, 1980.

As a result of the legislation discussed above, the National Bridge Inventory contains data for virtually all of the 260,000 bridges on the Federal-aid highway systems and for 98 percent of the 314,000 bridges on all other public roads.

The current requirements of the NBIS stipulate that bridges on public roads be reinspected and the inventory updated at least once every 2 years.

All inventories, inspections and appraisals are done under the direction of State and local governments. The inventory data is sent to the Federal Highway Administration (FHWA) at least once a year by the State for inclusion in the National Bridge Inventory (NBI). The NBI is maintained by the FHWA and used for many purposes.

2. PURPOSE OF BRIDGE INVENTORY AND INSPECTION

Bridge inventory data has many uses for both bridge owners and the nation as well. The first step in the rational management of any extensive array of physical elements is a current inventory of the condition, age, capabilities and needs of its component parts.

The primary purpose of the inspection program is to be sure that bridges are safe for public use. In accordance with the NBIS, all bridges must be evaluated for safe load capacity. If the bridge will not safely carry the maximum State legal load, it must be load posted or, if not safe to carry at least equal to 13.35 kiloNewtons (3 tons), it should be closed to traffic.

While there is a maximum national load limit on the Interstate highway system, individual States may set lessor load limits on the Interstate system and are free to set any load restrictions they deem appropriate on other public roads within their State. As of December 31, 1981, nearly 150,000 of the Nation's bridges were reported as requiring load restrictions. About 3,400 bridges were reported as closed to all traffic.

These figures are positive indications that State and local governments are using inventory and inspection data to limit the loads on sub-standard public bridges.

While the public safety is and must be assured through application of the provisions of the NBIS, the data gathered has many additional uses in the overall management of the highway system.

When the NBIS was initiated, many individuals and management officials of State and local highway agencies did not have a reasonable knowledge of how many bridges they owned, much less what condition they were in. Most forward thinking engineers and managers rightfully saw the NBIS as a means to meet their bridge system management needs.

From a more practical viewpoint, the NBIS provided State and local government officials the opportunity, for the first in many cases, to document the fact that bridge needs had reached the critical stage on many elements of the Nation's highway system. Many experts have been voicing this opinion for some time, but it has never been fully documented to the extent that national, State and local governmental bodies had to face the hard, physical facts that more resources must be devoted to bridges.

From the inception of the NBIS in 1971, many State officials elected to gather data over and above that required for NBIS purposes. Some of these will be discussed later. However, obvious purposes for the data include:

1. Establishment of physical records for each bridge.
2. Predictions of rate of deterioration.
3. Failure predictions.
4. Cost-effective evaluation of past bridge repair and rehabilitation.
5. Truck overload routing.
6. National defense uses.
7. Maintenance scheduling.
8. Establishment and documentation of historical bridges.

3. DATA GATHERING

From the inception of the national program, rigid standards were established for the qualifications of the inspectors and the format of the data for inclusion in the NBI.

According to the requirements of the NBIS, the individual in charge of the State bridge inspection organization must:[1]

1. Be a registered professional engineer; or
2. Be qualified for registration as a professional engineer under the laws of the State; or
3. Have a minimum of 10 years of experience and completed a comprehensive training course based upon the "Bridge Inspector's Training Manual."

STRUCTURE INVENTORY & APPRAISAL SHEET

Revised 12-78

IDENTIFICATION

State ____
Hwy District ____
County ____ City/Town ____
Inventory Route ____ On ☐ Under ☐
Features Intersected ____
Facility Carried by Structure ____
Structure No. ____ 1 of ____
Location ____
Min. Vert. Clearance, Inv. Rte. ____ ' ____ "
Milepoint ____
Road Section No ____
Defense Bridge Description ____
Defense Milepoint ____
Defense Section Length ____
Latitude ____
Longitude ____
Physical Vulnerability ____
By pass, Detour Length ____
Toll ____
Custodian ____
Owner ____
FAP No ____

CLASSIFICATION

Highway System ____
Administrative ____
Functional ____

	By	Date
Transfer of Data		
Maintenance Insp		
Condition Analysis		
Appraisal		
Cost Estimate		
General Review		

STRUCTURE DATA

Year Built ____
Lanes on Str ____ under ____
ADT ____ Year ____
Design Load ____
Appr Rdwy Width w/Sh'ld ____
Br Median ☐None ☐Open ☐Closed
Skew ____
Structure Flared ☐Yes ☐No
Traffic Safety Features ____
____
Navigation Control ☐Yes ☐No
- Vertical ____ ft
- Horizontal ____ ft
Open, Posted, or Closed ____

Type Service ____ (code)
Structure Type - Main ____
- Approach ____
No of Spans - Main ____
- Approach ____
Total Horiz. Clearance ____ ft
Max Span Length ____ ft
Structure Length ____ ft
Sidewalk ____ Lt ____ ft, Rt ____ ft
Br Roadway Width (curb-curb) ____ ft
Deck Width (out-out) ____ ft
Vert Clearance over Deck ____ ' ____ "
Underclearance - Vertical ____ ' ____ "
- Lateral Right ____ ft
- Left ____ ft
Wearing Surface ____

CONDITION

	Material	Condition Analysis	Rating (9-0)
Deck			
Superstructure			
Substructure			
Channel & Channel Protection			
Culvert & Retaining Walls			

Estimated Remaining Life ____ Approach Roadway Alignment ____
Operating Rating ____ Inventory Rating ____

APPRAISAL

	Deficiencies	Rating (9-0)
Structural Condition		
Deck Geometry		
Underclearances - Vertical & Lateral		
Safe Load Capacity		
Waterway Adequacy		
Approach Roadway Alignment		

PROPOSED IMPROVEMENTS

Year Needed ____ : Completed ____ Describe (item 75) ____
Type of Service ____
Type of Work ____
Improvement Length ____ ft
Design Loading ____
Roadway Width ____ ft
Number of Lanes ____ Prop Rdwy Improvement - Year ____
ADT ____ Year ____ - Type ____

Cost of Improvements ____ $ ____ ,000
Prel Engrg ____ $ ____ ,000
Demolition ____ $ ____ ,000
Substructure ____ $ ____ ,000
Superstructure ____ $ ____ ,000
Insp Date ____

Remarks:

Figure 1

An individual in charge of a bridge inspection team must meet the same qualifications or have a minimum of 5 years bridge inspection experience and have completed a comprehensive training course in bridge inspection.[1]

While many States collect more data than is required for NBI purposes, the standard data for the NBI is shown in Figure 1, "Structure Inventory and Appraisal Sheet." The data gathered must be in conformance with the "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges, January 1979."[2]

In accordance with the NBIS, each bridge must be inspected a minimum of once every 2 years; oftener if the condition of the bridge warrants more frequent inspections.

The data shown on the Structure Inventory and Appraisal (SI&A) Sheet is considered minimal for national purposes. (A shorter version is permitted for use by local governmental officials on bridges on less important highways.) There has been great pressure from many diverse sources to expand the number of required data items on the SI&A Sheets. For the most part, these pressures have been resisted to minimize the economic hardship that inspection requirements impose on State and local governments.

Further, while most States and bridge experts agree that a 2-year lapse between inspections is about the maximum that can be tolerated if the general safe public use of bridges is to be assured, evaluations are underway to determine if a longer lapse between inspections is appropriate for certain categories of bridges. Tentative results indicate that for bridges in very good condition that are not over waterways, a longer time lapse between inspections may be appropriate.

For the most part, the inventory, inspection and appraisal of the Nation's bridges has taken place with remarkably good results and at a better than expected pace. The 260,000 bridges on the Federal-aid highway system have virtually all been inspected and inventoried and 98 percent of the 314,000 bridges on all other public roads not on the Federal-aid highway system have been inspected and inventoried since these were included under the NBIS in March 1979.

4. A PLANNING TOOL

The immense amount of data in the NBI now can be used for a variety of purposes. For example, using the data in Figure 2, one can determine the correlation the age of bridges versus the percent which are deficient by decade. These plots indicate the general trend of the life span of bridges, emphasize that bridges off the Federal-aid highway system become deficient faster than those on the system and that the life span of off-system bridges is shorter than those on the system.

Figure 3 depicts the number of bridges still in use by decade built. Again there is much useful data, The "boom" and "bust" years of bridge building can readily be seen to be influenced by the general national economy. One can also see that there are a substantial number of bridges built before 1940 still in everyday use. Plans must be made to accommodate heavy maintenance, rehabilitation or replacement of most of these during the next decade or two, if the highway system is to remain useable.

From a national viewpoint, the available NBI data is used by the Secretary of Transportation and the Congress in establishing national transportation policy. At the moment, it is one major factor in deliberations pertaining to whether there should be a uniform national truck size and weight law, and, if so, what should national weight limits be. For example, even though most of the Interstate highway system was designed to accommodate HS20 trucks as designated by the American Association of State Highway and Transportation Officials, more than

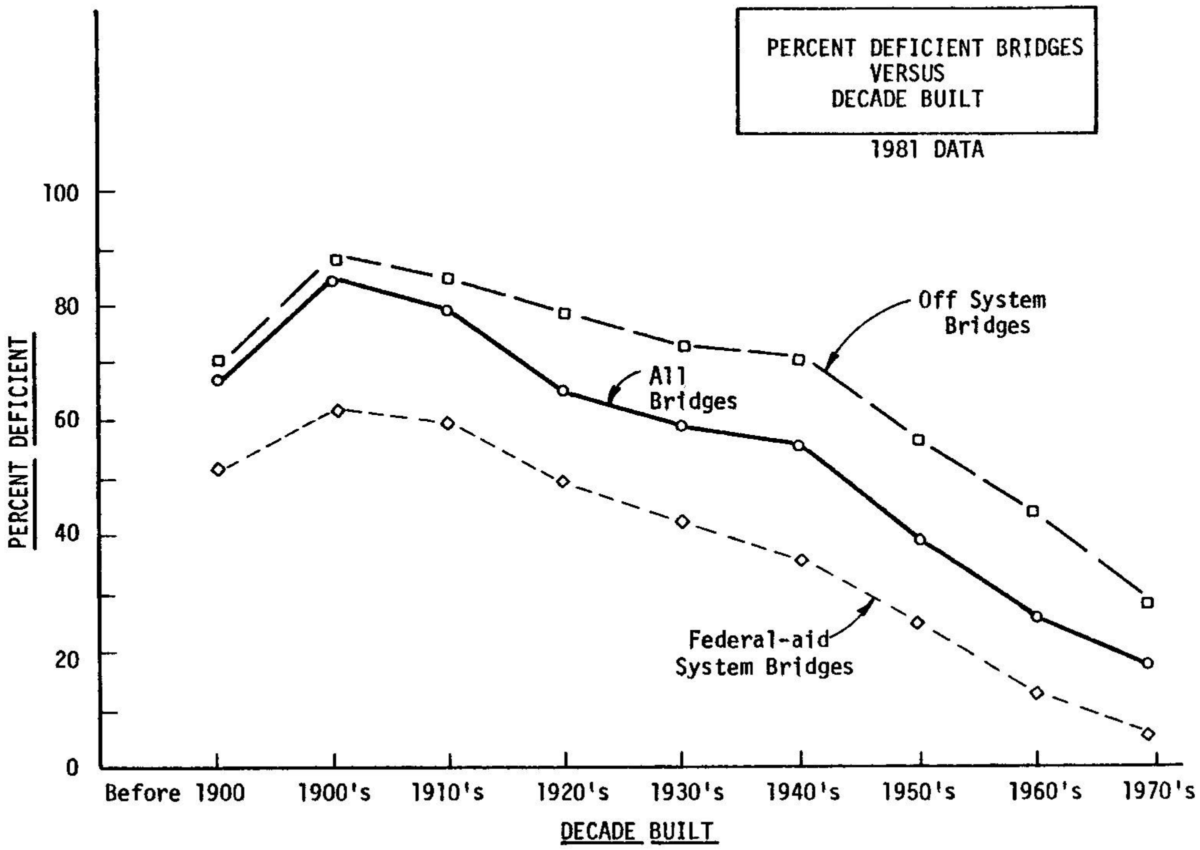

FIGURE 2

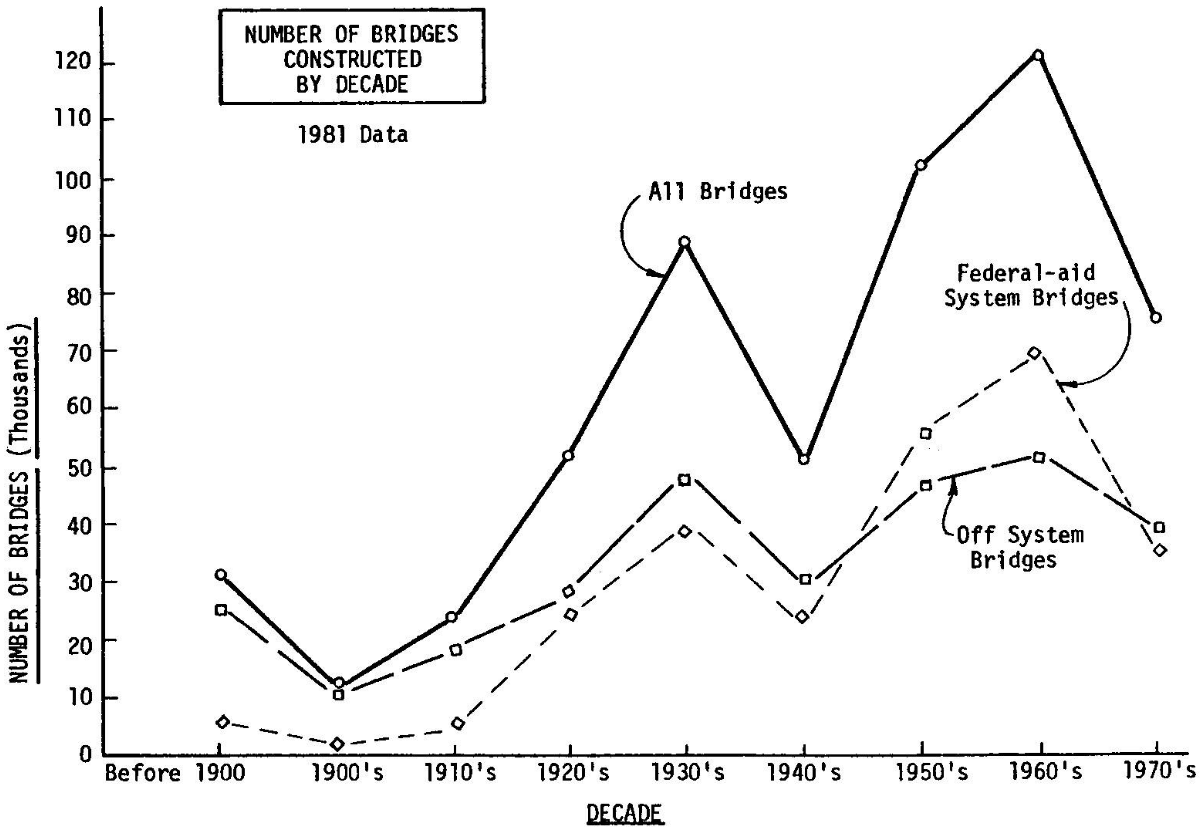

FIGURE 3

800 Interstate bridges have only H15 load capacities.[3] Fortunately, most of these are not main line structures, but they must be accounted for in any deliberations pertaining to national truck size and weight issues.

One critical element of much of the Federal-aid highway system is that of concrete bridge decks. It is well known that most States in northern and moderate climates are having problems caused by deicing chemicals applied to concrete bridge decks. But it was only by use of NBI data that the true magnitude of the bridge deck problem was identified. More than 39,000 bridges have a deck condition rating of 4 or less (marginal condition).[4] The deck data is being used to formulate a new national policy regarding cost-effective methods to prevent further deterioration.

Using NBI data, it was established this past year that bridge resurfacing, restoration, rehabilitation and reconstruction needs on the Interstate highway system are $9.6 billion.

Perhaps the most important use of the NBI data is to track bridge needs versus the national bridge program size. Figure 4 illustrates this. Caution in using the data is advised. One reason for the rapid growth in bridge needs depicted on the chart is the fact that the total bridge inventory was not essentially complete until the very last year or so shown. Earlier need estimates were based upon partial data.

This year, a new historic bridge item is being added to the required NBI data. Because special permits, project approval processes and reports must be developed and approved before historic bridges are replaced or rehabilitated, it is important to know their historicity well in advanced of any project. Otherwise, intolerable and costly delays in projects involving historic structures may result. It is, of course, beneficial to know the categories of existing historical bridges whether or not improvement projects are contemplated.

The NBI is used to supply certain data to the Department of Defense to enable adequate planning for national emergencies. These uses are obvious.

In the last several years, some experts have suggested that the NBI should contain data on bridges with fracture critical members. (A fracture critical member is one whose failure will cause failure of the bridge. Examples are the ties of tied arches, suspension bridge cables, tension chords of trusses, etc.) This knowledge is perhaps important for States to keep track of but is not at this time considered vital to the NBI.

While bridge structural safety is of critical importance, in an average year as many as 1,000 to 1,500 people are killed in the United States when errant vehicles strike bridge members. Bridge rails, guardrails, piers, end posts and abutments are the most common components hit. For this reason, a portion of the NBI is devoted to the adequency of safety appurtenances on bridges. These data are used by many experts in assessing safety, establishment of programs and setting priorities for the upgrading of bridge rail, guardrail and other safety devices on bridges. For example, the authors of NCHRP Report 239, "Multiple-Service-Level Highway Bridge Railing Selection Procedures," used NBI bridge railing data.[5]

5. A FINANCIAL TOOL

One of the most significant planning tools the NBI data can be used for is that of planning the financial requirements for development of natural resources or new manufacturing or processing centers. The existing load capacity of bridges and potential costs to increase them to desired amounts can be readily determined.

With rapidly changing sizes and characteristics of both passenger vehicles and trucks, the NBI data can be used to evaluate the need for and impact of any change in geometric standards for various elements of the existing national highway system.

Of primary importance is the capability of using the NBI data to evaluate energy impacts on the national economy stemming from current or future detours of heavy trucks. Nearly 127,000 of the Nation's bridges are currently reported as structurally deficient. Annual detour costs well in excess of $1 million per bridge have been reported for heavily traveled bridges in urban areas.[6]

The NBI has been used from the very beginning of the program as a tool for prioritizing bridge replacement and rehabilitation projects. A sufficiency index for each deficient bridge is computed using the NBI data. Generally, the lower the sufficiency rating, the higher the priority for replacement. However, other local and State factors must be considered. The sufficiency rating is described in Appendix A of reference 2.

The NBI is useful as a means of selecting research projects, determining the need for innovative rehabilitation techniques and for budgeting of future research programs.

COMPARISON BETWEEN FUNDING LEVELS AND NEEDS FOR THE HIGHWAY BRIDGE REPLACEMENT AND REHABILITATION PROGRAM

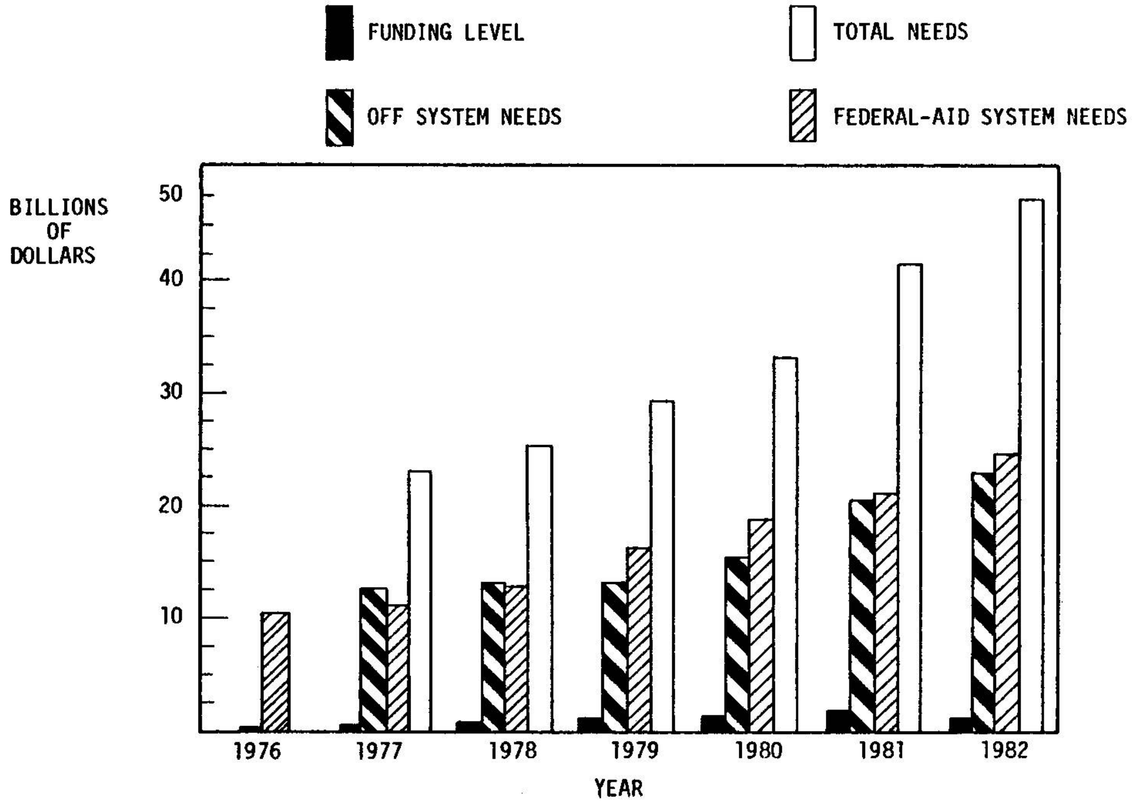

FIGURE 4

6. INSPECTION COSTS

The costs of inspection are important to evaluate the cost effectiveness of the program. Initial inventory and inspection costs have been reported as averaging from $43 to $650 per bridge. One State has reported an average cost of $0.67 per square meter ($0.06 per square foot). It is estimated that the average cost for initial inspection and inventory was about $300 per bridge. Because the initial inspection and inventory of a bridge often requires the measurement and recording of physical data, especially when construction plans are not available, subsequent inspections should be substantially less expensive.

The benefits resulting from the inspections substantially exceed the costs. These benefits accrue from prevention of loss of lives from bridge failures, prevention of detours and traffic delays imposed by closed bridges, savings of energy and general societal costs not incurred because bridges are available in time of fire, accident or related emergencies. Many times, the prompt discovery of a bridge defect may allow correction for a modest expense before major elements of a bridge fail. Computational analysis by the Federal Highway Administration staff has indicated that the benefits are in excess of $100 million per year.

7. SUMMARY

In summary, it can be demonstrated that:

1. A rational national bridge program depends upon an accurate and comprehensive bridge inventory.
2. The Federal Highway Administration holds national bridge data requirements to a minimum, but State and local officials can and often do add data items for their own management and planning purposes.
3. The benefits and savings in human life resulting from the bridge inspection program far outweigh program costs.
4. Other financial benefits are very large.
5. No State is yet taking full advantage of the bridge data available.

8. REFERENCES

1. U.S. Code of Federal Regulations, Highways, Title 23, Section 650.3.
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