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SESSION 3

Rehabilitation and Repair of Bridges

Modification et réparation des ponts

Sanierung und Reparatur von Brücken

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Experiences with Orthotropic Steel Deck and Cable Stayed Bridges

Expériences avec tabliers en dalle orthotrope et ponts à haubans

Erfahrungen mit orthotropen Fahrbahnplatten und Schrägseilbrücken

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SUMMARY
A survey is made of some important aspects related to maintenance, repair and rehabilitation of orthotropic steel deck and cable stayed steel bridges. Thermal effects due to machine laid resurfacing of orthotropic steel decks can no longer be neglected. Information is presented as an aid to diagnose such effects. The need for inspecting cables and for provisions for replacement of cables is emphasized, and methods for improved corrosion protection of cables are briefly described.

RESUME
Le rapport présente quelques aspects importants de l'entretien et de la réparation de dalles orthotropes et de ponts à haubans. Des effets thermiques lors d'un renouvellement du revêtement ne peuvent plus être négligés. Des informations sont données pour faciliter le diagnostic de ces effets. La nécessité d'inspecter les câbles et de prendre des précautions pour les remplacer est soulignée. Des méthodes de protection anticorrosion améliorée des câbles sont décrites brièvement.

ZUSAMMENFASSUNG
Dieser Beitrag gibt einen Überblick über einige wesentliche Aspekte im Zusammenhang mit Unterhaltung und Reparatur von Brücken mit orthotroper Fahrbahnplatte und seilverspannten Stahlbrücken. Temperaturauswirkungen infolge Belagserneuerungen mit Fertigern können nicht mehr vernachlässigt werden. Dazu werden Angaben gemacht. Die Notwendigkeit einer Überprüfung der Seile und einer Seilauswechselung wird unterstrichen und Angaben für einen verbesserten Korrosionsschutz der Seile gemacht.

1. INTRODUCTION

1.1 Brief historical Background

Orthotropic steel bridge decks have been built since about 35 years, modern cable stayed bridges since about 25 years. In Germany, the first orthotropic steel bridge deck, with reinforced concrete as wearing surface, had been built in 1947. All orthotropic bridge decks built from 1950 have a bituminous wearing surface with an insulating layer of sufficient adhesive and waterproofing qualities.

The basic principle of cable stayed bridges is not at all new. Plans of cable stayed bridges are known from the 17th century. Failures and collapses have apparently hampered further development. Insufficient tensile strength of the materials used, faulty detailing of connections and insufficient vertical and horizontal stiffnesses have certainly contributed to have cable stayed bridges let fallen into oblivion. The development of cables of sufficiently high tensile strength suitable for bridge construction and modern means of structural analysis finally met requirements needed for modern cable stayed bridges.

First designs of modern cable stayed bridges found no acceptance for the time being short after World War II. Today, it may be considered as a curiosity that in 1956 a bridge over the Rhine, with a main span of 256 m, had been erected with the aid of stay-cables but the bridge in it's final state was not a cable stayed bridge. However, the final "break-through" came about shortly afterwards and cable stayed bridges were built in short succession.

1.2 Statement of the Problem

Both, the orthotropic steel deck and the cable stayed bridges have gone through several principal phases of development until they have reached the present state. Naturally, at the begin of the development phase problems such as refinement of analytical methods, shop fabrication, welding and erection techniques had top priority. From the beginning, due consideration was given to the problem of surfacings for orthotropic steel decks. However, the relation between the structural elements of the orthotropic plate regarding their relative stiffnesses and the bridge deck surfacing and the waterproofing was not fully recognized. Furthermore, to what extent the micro-climate affects corrosion of stiffeners was only learned by experience. Economical considerations brought about the change from hand to machine laying of bituminous surfacings. Thermal effects resulting from machine laying have been not always allowed for in the original design of the bridge.

The full-locked-spiral-cable, the type of cable almost exclusively used in Germany for cable stayed bridges, was believed to be completely sealed internally and, with the outside painted, to outlast the assumed life expectancy of the bridge. After about one decade of service in an increasingly worsening environment such an assumption proved to be too optimistic. The extent of the overall corrosion problem including the stress-corrosion problem was again only learned by experience after several years of service.

1.3 Scope of the Report

This report surveys aspects related to maintenance, repair and rehabilitation of orthotropic steel deck and cable stayed steel bridges. A few cable stayed concrete bridges have been built recently and probably more are to come in near future. Because of their short service life, not enough experience could be gained so far. In order to achieve brevity in this report, detailed information on bituminous surfacing and paint systems has been omitted.

Aspects related to the orthotropic steel deck are described in Chapter 2, while aspects related to cable stayed steel bridges are reviewed in Chapter 3. Conclusions and recommendations are listed in Chapter 4.

2. ORTHOTROPIC STEEL DECK

2.1 Present State of Development

The present "standard" orthotropic plate has a 12 mm thick deck plate stiffened with longitudinal trapezoidal stiffeners, 300 to 325 mm wide at 300 to 325 mm transverse spacing. The webs of cross-beams are cut out so that stiffeners are continuous for the full length of the bridge with shop, respectively, field splices at or near the points of contrafluxure. The stiffeners are continuously welded to the deck plate and supported at the cross-beams by web to web stiffener to cross-beam welds. Thicker deck plates and deeper stiffeners are normally only used where structurally necessary, usually in the region of peak negative moments. Such "standard" orthotropic plates show an overall satisfactory behaviour, shop fabrication and assembly are likewise nearly "standardized" procedures and the even surfaces favour relativ easy paint application.

Orthotropic plates, even those with torsionally stiff stiffeners, have a poor transverse load distribution. Results of strain gauge measurements on the bottom flange of stiffeners of a "standard" orthotropic plate are shown in Fig. 1 [1]. Studies made under traffic in motion show that about seventy to eighty per cent of all wheels of commercial vehicles passed within about ± 300 mm of the centre line of the "ideal" wheel track. These two facts combined indicate that orthotropic plates have heavily loaded and less heavily loaded stiffeners.

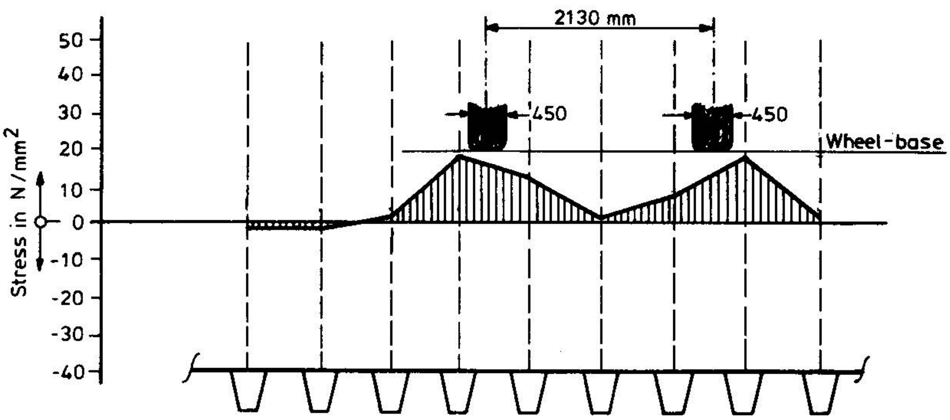


Fig. 1 Transverse Load Distribution

2.2 Inspection

The development of an appropriate procedure and sequence for the inspection of orthotropic steel decks is important in order to ensure the degree of thoroughness and completeness in an inspection that is essential. The scope of inspection may be described in terms of its scale or intensity and its frequency as follows:

- Superficial inspection carried out quickly and frequently by highway maintenance personnel, not necessarily trained in bridge inspection. The purpose of superficial inspection is to report the fairly obvious deficiencies of the bridge deck which might lead to traffic accidents or cause high maintenance costs if not treated promptly.
- Principal inspection is made by a trained inspector under the general supervision of a bridge engineer. This type of inspection usually falls into two categories referred to as general and major defined by frequency and intensity. The general inspection will be primarily a visual inspection supplemented by standard instrumented aids and will entail examination of the bridge deck and the underside of the orthotropic plate. A written report will be made of the condition of the orthotropic plate and its various elements. The major principal inspection will be more intensive and require close examination of all elements, involving setting up of special access facilities where necessary. The intervall between major inspections will vary between 5 to 7 years. A full report containing photographs, drawings, etc. will usually be prepared. A crew whose sole function is to make detailed principal inspection should have a well equipped vehicle stocked with hand tools, with some scaffolding, traffic signs, safety equipment and non -destructive testing equipment for weld inspection.
- Special inspection is made in connection with unusual circumstances, such as exceptional loadings, with occurance of major weaknesses or with reassessment of the orthotropic plate after major repairs. Such inspections may require a good deal of supplementary testing and structural analysis and will invariably require detailed involvement of a bridge engineer and, in some cases, of a specialist.

2.3 Cracks

Cracks in the steel deck plate, in stiffeners and cross-beams have found to be a rare exemption. Cracks have been found especially in the stiffener to cross-beams welds. Fabrication tolerances and welding sequence appear to influnce possible cracking in this welded detail which may be prone to fatique cracking. Some cases are known where rusting took place in closed stiffeners that are nominally sealed by welding.

2.4 Corrosion

The need for a good, durable and longlasting corrosion protection of the deck plate surface is obvious. Not so obvious is, at first sight, the need for a high class corrosion protection of the underside of an orthotropic plate. The temperature gradients, inherent to orthotropic plates, cause condensation which in turn causes rusting. Dewdrops on the underside of a stiffener and condensation stains on the stiffener web are shown in Fig. 2. Experience has clearly demonstrated that only the best paint systems available are capable to give the desired corrosion protection. Since condensation is a permanent occurrance, intervals between application of paint layers should be kept as short as possible.

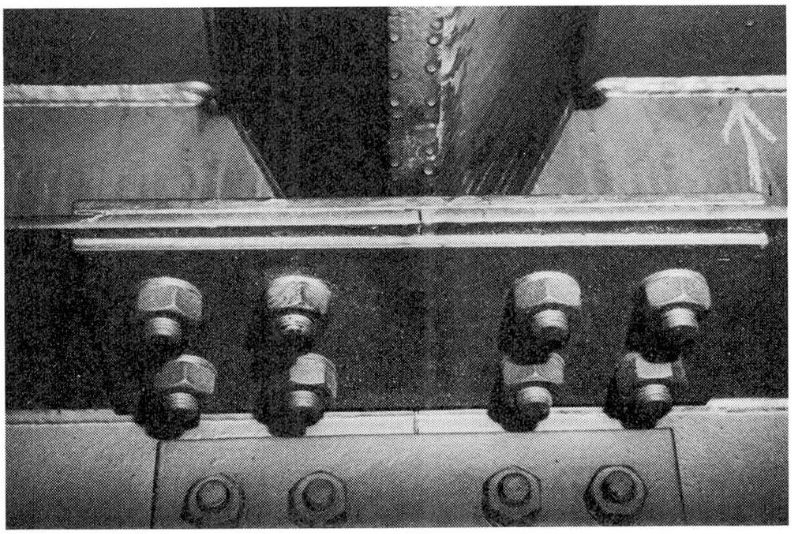


Fig. 2 Dew-Drops on the Underside of Stiffeners

2.5 Connection between Plate Siffness and Surfacing and Waterproofing

Cracking of the surfacing may loosely be grouped under the two categories of cracking over elements much stiffer than the orthotropic plate, e.g. webs of main girders, box girders, stiffening girders, etc. and cracking due to insufficient bond to the insulating layer or to the steel plate. When cracks have formed, water may penetrate and this causes corrosion of the steel which gradually spreads under the surfacing away from the cracks. This results in loss of adhesion which, in turn, leads to the formation of a series of parallel cracks as demonstrated in Fig. 3.

Fig. 3 Severe Cracking of the Surfacing

Ongoing research in Germany into the interaction of plate stiffness factors and surface cracking suggests that it may be advisable to formulate stiffness criteria for orthotropic plates in addition to the traditional strength requirements to achieve a smoother transition between unfavourable stiffness ratios.

2.6 Machine Laying of Surfacings

The service life of surfacings for orthotropic steel decks varies between about 8 and 18 years, excluding of course faulty surfacings or unsuitable material. Consequently, a bridge will have to be resurfaced several times during its service life.

In Germany, "gussasphalt" is the favoured surfacing material used. Gussasphalt is similiar to mastic asphalt which has been used in the case of Severn, Forth and Humber bridges in the U.K.

Traditionally gussasphalt and mastic asphalt have been laid by hand, because they remain fairly fluid for some time after laying and they are also quite dense, so that they tend to trap air within it. The hand trowelling operation would correct the tendency for levels and thicknesses to change after laying and also encourage the escape of trapped air to reduce the risk of blisters and holes forming in the asphalt. However, hand laying is slow, it may give irregular surface, and the number of craftsmen who can do it well is diminishing. Consequently there has been increasing pressure to lay asphalt surfacings by machine to economize on the laying operation and to minimise delays to traffic. Provided necessary precautions are taken, asphalt surfacings can be machine laid successfully. Requirements for a successful machine laying are absolutely dry and clean surfaces free from any oily substances. If those requirements are not observed, the risk of blisters forming in the asphalt and insufficient bond to the surface must be faced with all its consequences.

As already mentioned above, hand laying is a slow operation. Consequently thermal effects were localized and, as a rule, only paint was affected by the heat of the laid asphalt. Thermal effects in structural elements could be neglected. The average machine laying speed is between 1.4 and 2.0 m/min. The minimum width of the spreaders used for the machine laying operation corresponds to the width of one traffic lane (in Germany, that would be about 3.5 m). Thermal effects on the structure can no longer be neglected.

The results of operational research in Germany [2] suggested that existing structures be investigated into thermal effects of machine laying [3] . Four bridges were investigated representing the types of steel bridge most common to modern orthotropic steel deck bridges: plate girder bridge with parallel flanges, haunched plate girder bridge, box girder bridge (curved) and cable stayed bridge. The basic assumption for all four bridges investigated was resurfacing of one half of the total bridge deck under traffic. The investigation was aimed at getting better knowledge of thermal effects on bending stresses and horizontal loads.

The following assumptions were made for the investigation:
- Dead load of the completed structure including weight of the surfacing for one half of the bridge deck, i.e. surfacing on the other half had been taken away in preparation of resurfacing;
- Full design live load for the remaining traffic lanes;
- Design wind load;

- The proportional weight of the machine laid surfacing layer;
- The live load of vehicles and machines needed for the machine laying operation;
- Thermal effects due to machine laying operation.

The results obtained, in comparison to the original design, are as follows:

- Stresses may reverse in some cross-sections, i.e. compression stress in the original design are reversed to tensile stresses;
- Tensile stresses due to the effects of machine laying amount to between 20 per cent and 28 per cent of the total resulting tensile stress. Total resulting tensile stresses can exeed allowable tensile stress up to 16 per cent.
- Compression stresses due to the effects of machine laying amount to between 64 per cent and 81 per cent of the total resulting compression stress. Total resulting compression stresses can to a considerable amount exeed allowable values.
- The stress ratio

  $$\frac{\text{maximum stress due to machine laying}}{\text{maximum original design stress}}$$

  is for tensile stresses 1.00 to 1.31, for compression stresses 1.79 to 3.26. The higher values for compression stresses are partly due to stability requirements. However, instability is normally not a serious problem for orthotropic plates.
- Horizontal loads can be appreciably higher than design horizontal loads.

The investigation clearly demonstrated that reliable results can only by expected if the calculations are based on the actual temperature distribution in the structure. Simplified models of temperature distributions may give incorrect results.

It should be noted that no structural failure in bending due to machine laid surfacings has been observed so far. However, the results of the investigation suggest that thermal effects due to machine laying may partially use up safety reserves which, fortunately, most structures have.

Table 1 shows increase factors f for horizontal loads

$$f = \frac{\text{horizontal load due to machine laying of surfacing}}{\text{horizontal load due to design assumptions}}$$

at the supports of the bridges investigated. The guidebearing on the abutment of the bridge with a calculated increase factor f = 263 sheared off completely from the bearing seat. This may be an indication that safety reserves in a structure are not always inexhaustible.

2.7 Temperature Range of Deck Plate and Insulating Layer

Orthotropic Bridge decks require, as already mentioned above, the best possible corrosion protection of both the deck surface and the underside. Regardless of the base material used for paint systems, e.g. oil, resin, polythene, etc., the degree of temperature is limited which paint systems can endure without damage. In Germany, the material used for insulating layers has changed from pure bituminous material to tar-epoxy-resin which is believed to have better adhesive qualities than pure bituminous material such as mastix.

Plate temperatures are useful as starting point for calculating temperature distributions in a steel bridge and as criterion for

$$f = \frac{\text{horizontal load due to machine laying of surfacing}}{\text{horizontal load according to design assumptions}}$$

= increase factor against horizontal design load

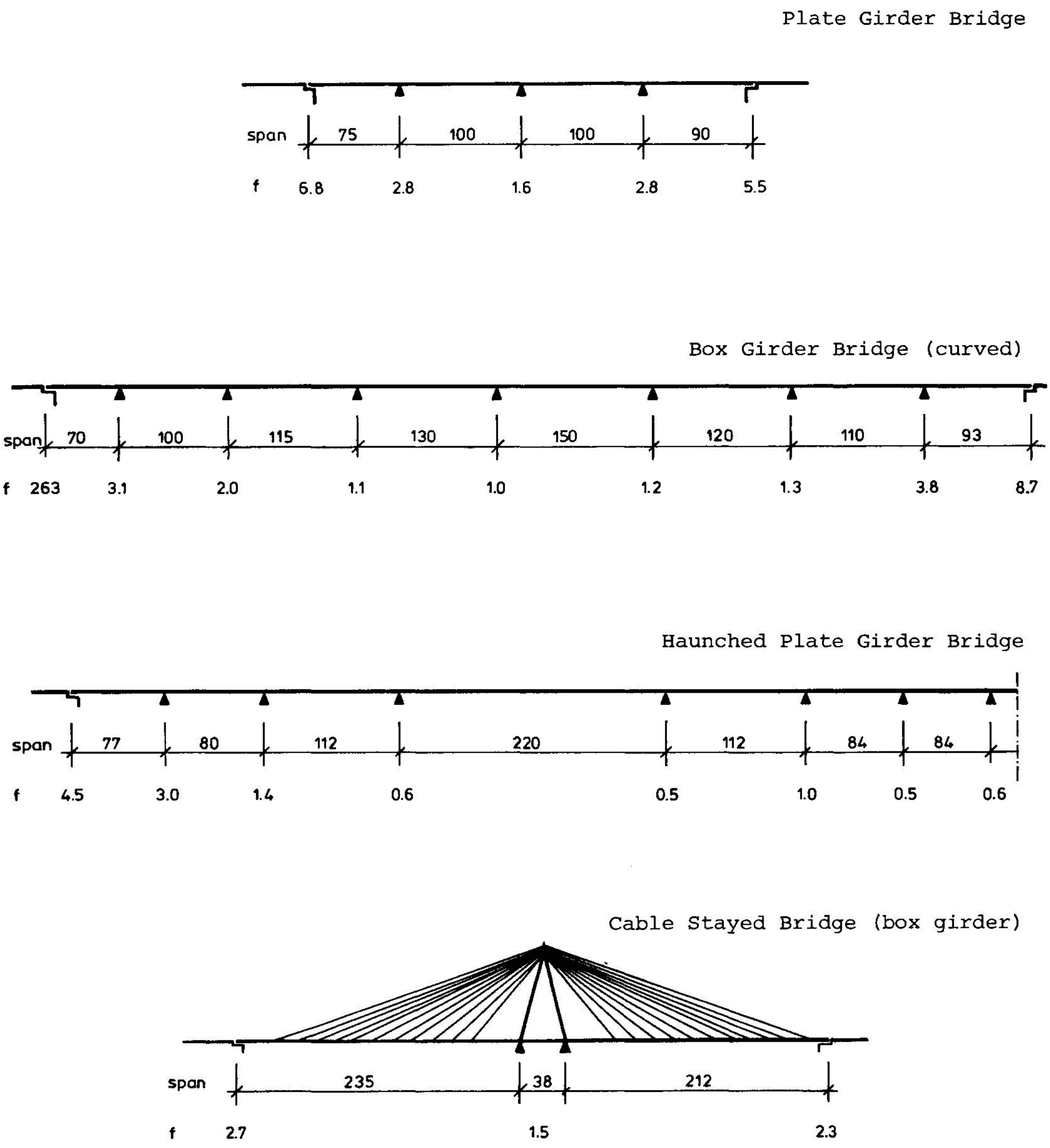


Table 1 : Horizontal loads induced by machine laying of bituminous surfacings on orthotropic steel bridge decks.

heat resistance of paint systems. The maximum plate temperature and the duration of certain temperature ranges, both values are interrelated, define the criterion for heat resistance of paint systems. The results of many temperature measurements in various bridges under various climatic conditions and with varying surfacing systems can be used to predict the maximum deck plate temperature and the duration of temperature ranges. The results are presented in Fig. 4 to 7.

Fig. 4 to 7 may be used as guide to questions arising in connection with resurfacing such as:

- Possibility of damage to existing paint;
- Maximum allowable mixing temperature of surfacing material in order to avoid damage to existing paint, e.g. when there is a choice between surfacing systems;
- Choice of paint system capable to withstand temperature effects without damage, e.g. when only a certain surfacing system has to or can be used;
- Formulation of requirements for corrosion protection systems;
- Prediction of maximum deck plate temperature as starting point for calculating temperature distributions.

Resurfacing may require, according to the condition of the existing surfacing, a total resurfacing, i.e. resurfacing will have to start from the steel deck plate, or a partial resurfacing, i.e. resurfacing of the wearing course only.

From measurements taken the maximum deck plate temperature was empirically found to be

$$t_{max} = \text{variable } (t_S - t_P) + t_P$$

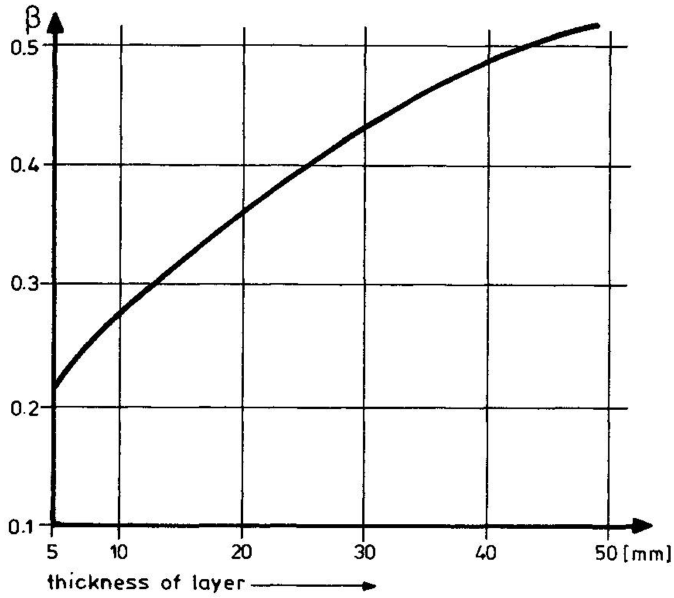
and the maximum surface temperature of the insulating layer

$$t_{max} = 0.44\, t_S + t_P$$

where the variable β is to be used in the case of total resurfacing as presented in Fig. 4 and the variable γ for the case of partial resurfing as presented in Fig. 5; t_S is the recommended mixing temperature of the surfacing material and t_P the existing deck plate temperature at time of laying. The β-values account for an existing layer of a few millimeter thickness. The use of Fig. 4 and Fig. 5 are best illustrated by an example:

- A typical resurfacing operation in Germany would have a first layer of "gussasphalt" with an average thickness of 35 mm laid on an insulating layer about 2 mm thick. The recommended mixing temperature t_S for "gussasphalt" is 250°C. The deck plate temperature t_P shall be 40°C, a value typical for the months May and September. From Fig. 4 a β-value = 0.46 is taken for a thickness of 35 mm. With the above values the predicted maximum plate temperature would be

$$t_{max} = 0.46\ (250 - 40) + 40 = 137°C$$



$$t_{max} = \beta(t_S - t_P) + t_P$$

where

t_{max} = maximum temperature at underside of deck plate

t_S = recommended mixing temperature of surfacing material, e.g. t_S = 250°C for "gussasphalt"

t_P = deck plate temperature at time of laying

β = variable

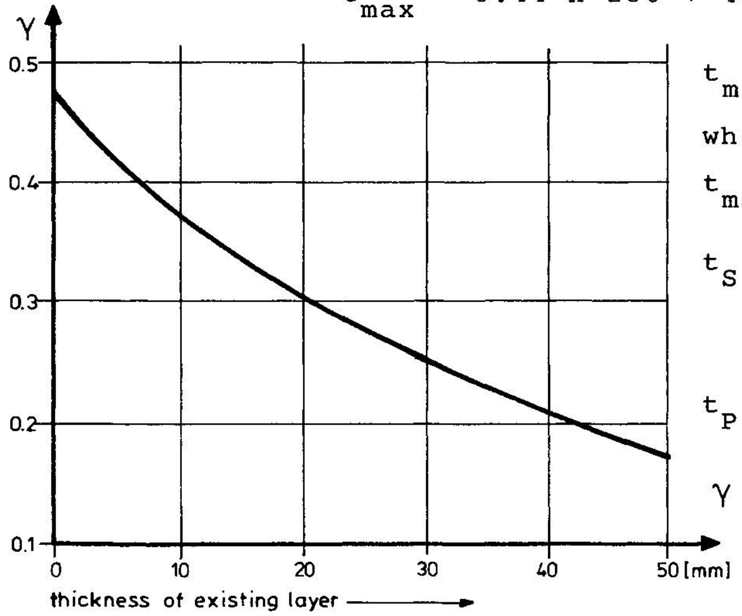
Fig. 4 Maximum Deck Plate Temperature due to Machine Laying of first Layer.

- A typical partial resurfacing operation in Germany would have a wearing course of "gussasphalt" with an average thickness of 35 mm laid on an existing layer about 35 mm thick. The deck plate temperature shall be 40°C. From Fig. 5 a γ-value = 0.23 is taken for an existing thickness of 35 mm. With the above values the predicted maximum plate temperature would be

$$t_{max} = 0.23\ (250 - 40) + 40°C = 88°C$$

- For a typical total resurfacing operation with the condition and values mentioned in the example for Fig. 4, the predicted maximum surface temperature of the insulating layer would be

$$t_{max} = 0.44 \times 250 + 40 = 150°C$$



$$t_{max} = \gamma(t_S - t_P) + t_P$$

where

t_{max} = maximum temperature at underside of deck plate

t_S = recommended mixing temperature of surfacing material, e.g. t_S = 250°C for "gussasphalt"

t_P = deck plate temperature at time of laying

γ = variable

Fig. 5 Maximum Deck Plate Temperature due to Machine Laying on already Existing Layer.

Fig. 6 can be used to predict the duration of certain temperature ranges for the paint on the underside of the deck plate and Fig. 7 for the insulating layer. The use of Fig. 6 will be illustrated by the following example.

Because the critical temperature range for normally used paint systems starts from 80°C, the duration of the temperature range above 80°C shall be predicted. The following data are needed: the recommended mixing temperature t_S, the thickness of the surfacing layer and the existing deck plate temperature t_P.

A layer of 35 mm thick "gussasphalt" is to be laid at a deck plate temperature t_P = 35°C.

In Fig. 6, the temperature range is plotted versus duration (time in min). The temperature range is the difference between an upper and a lower temperature limit. The upper limit is $t_{max} - t_P$ as shown in Fig. 4, the lower limit is the temperature above which the temperature range shall be predicted minus t_P.

In the example, the upper limit is 0.46 (250 - 35) = 100°C, where β= 0.46 is taken from Fig. 4, and the lower limit is 80°C - 35°C = 45°C. Consequently, the temperature range is 100°C - 45°C = 55°C as indicated by arrows in Fig. 6.

The predicted duration for a temperature range above 80°C is for:
the minimum duration 10 min + 75 min = 85 min
the average duration 10 min + 95 min = 105 min
the maximum duration 10 min + 110 min = 120 min

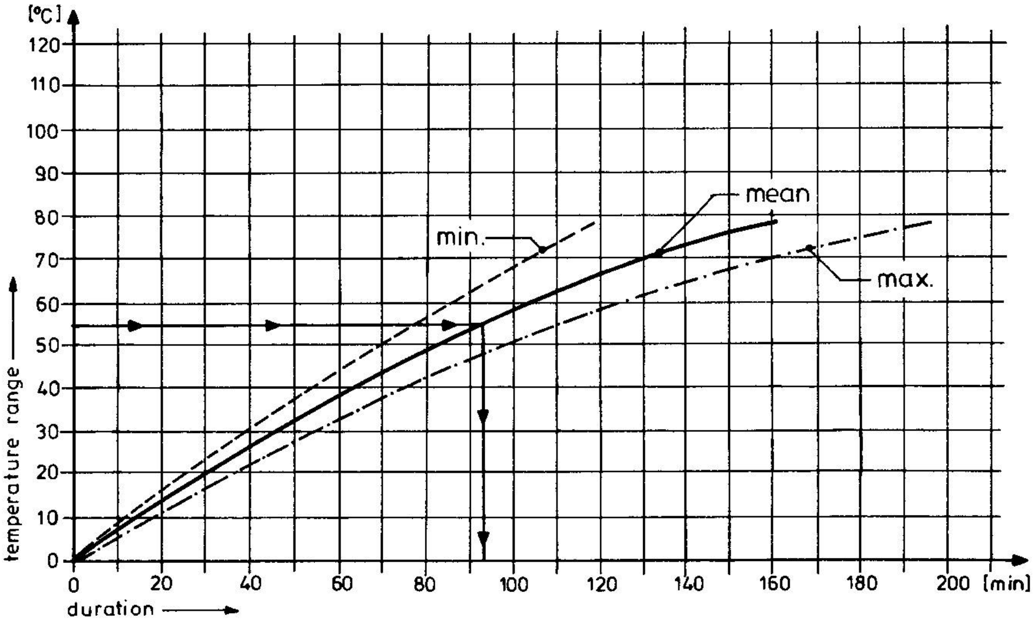


Fig. 6 Duration of Temperature Ranges for Paint Systems

The use of Fig. 7 will be illustrated by a following example. Critical temperatures for tar-epoxy-resin start from above 100°C. The recommended mixing temperature and the existing surface temperature of the insulating layer must be known in order to predict the desired temperature range.

In the example, a layer of 35 mm thick "gussasphalt" is to be laid on an insulating layer with a surface temperature of 35°C. The duration of the temperature range above 100°C shall be predicted.

Required is again the upper and the lower temperature limit. The upper limit is 0.44 x 250°C = 110°C, the lower limit is 100°C - 35°C = 65°C. Consequently, the temperature range is 110°C - 65°C = 45°C as indicated by arrows in Fig. 7. The predicted average duration for temperatures above 100°C is about 60 min.

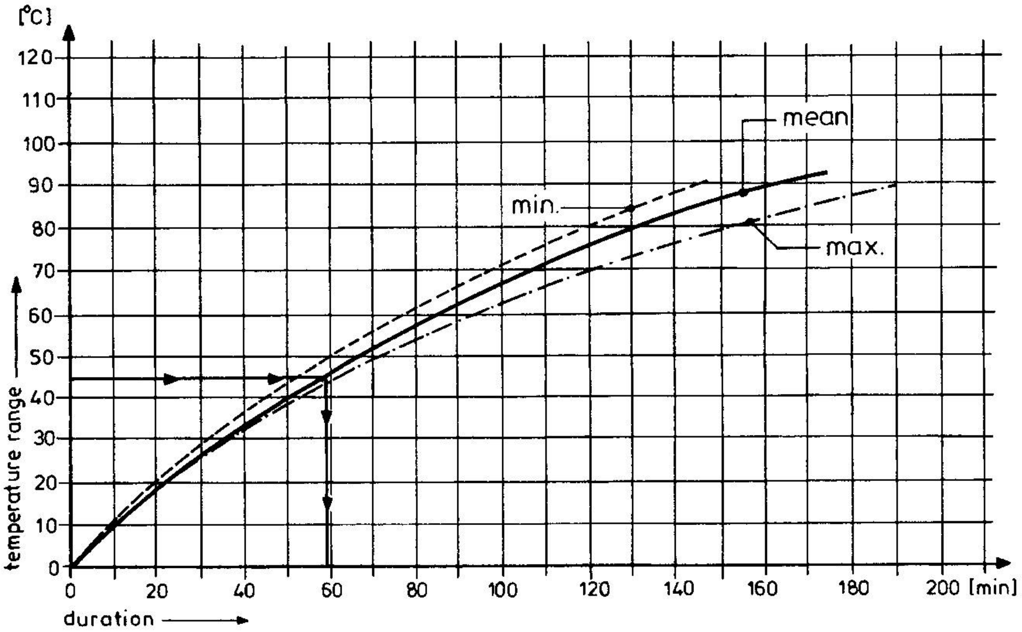


Fig. 7 Duration of temperature Ranges for Insulating Layers

3. CABLE STAYED BRIDGES

3.1 Present state of Development

The present state cable stayed steel bridge uses single cable stays normally either in the form of radiating fan cable stays or harp cable stays depending on bridge aesthetics or structural requirements or a combination of both, aesthetics and structural requirements. Main spans vary between about 175 m and 350 m. Normal span to main girder depth ratios may vary between about 60 and 100. Main girders are either plate girders or box girders with an orthotropic steel plate deck. Cable supports along the bridge centre line require a box girder type main girder. Towers may be either vertical or A-shaped. The type of cable, almost exclusively used in Germany, is the full-locked-spiral-cable. Cable stayed bridges are normally very flexible to aesthetical, structural and geometrical requirements.

3.2 Cable Stays

3.2.1 Wire Strands

From the point of view of tensile strength a wire with the highest possible tensile strength appears to be desirable. From the point of view of durability a wire having the least sensivity appears to be desirable. Unfortunately, an economically feasible material for wire strands to be used in full-locked-spiral-cables combining both qualities does not exist at present. A practical compromise has to be found between tensile strength and durability requirements. Wires with a tensile strength not exeeding about 1600 N/mm^2 are believed to be a practical compromise between required tensile strength and sensivity to corrosion and stress-corrosion.

The "standard" full-locked-spiral-cable is made up of two different wire profiles. Round wires represent the core, Z-shaped wires enclose the core and form the outer layers. All wires in a cable must have the same tensile strength.

3.2.2 Transport of Cables

Full-locked-spiral-cables are prefabricated in the shop to the required lenghts and shipped to the site on reels. Although the diameter of the reel is, in most cases, restricted by clearence limitations along the route, its diameter should not be less than about 35 to 40 times the diameter of the cable. Experience has demonstrated that cables transported on reels having smaller diameters tend to show a permanent "wave-forming" despite of pre-stretching. The "wave-lengths" correspond to the length of the coil around the reel used for the transportation of the cable. Such "waves" may affect the total resulting effective modulus of elasticity of the cable.

3.2.3 Use of Single Cable-Stays

In some of the "older" cable stayed bridges up to 13 full-locked-spiral-cables, erected one at a time, had been connected by special cable bands to form the "main" cable. Each of the cables is, of course, individually socketed and anchored. Grooves formed by the round cables had been puttied up in order to seal the interior of the "main" cable. With the outside painted, the system was believed to give a satisfactory, long lasting corrosion protection. However, this system fell short of the designer's expectations. The "inside" cables can neither be inspected nor re-painted. In addition, erection of cable stayed bridges proved to be easier and more economical, after having gained more experience, if single cable stays at shorter intervals are being used. The system of "main" cables for cable stayed bridges can no longer be recommended.

3.2.4 Cable Replacement

For a period of about 10 to 15 years, bridge designers were convinced, as already mentioned, that full-locked-spiral-cables are a long lasting structural element and would in all probability outlast the expected service life of a cable stayed bridge.

Possible fatigue was already a point of concern from the moment of first designs of cable stayed bridges. Extensive testing of wires and full sized test specimen of full-locked-spiral-cables suggested that fatigue due to live load would, in all probability, not shorten the expected service life of cables. Extensive measurements in a cable stayed bridge in service [4] appear again to support the thesis that fatigue damage due to "static" live load is not to be expected.

Whilst "static" live load appears to be not a serious problem regarding fatigue life expectancy, the same cannot be said for "dynamic" effects, i.e. the sum of all dynamic effects including wind, if at the same time corrosion takes place. Past experience suggests that the combination of dynamic effects and corrosion may present a serious problem to cables especially at or near the anchorage.

Cables had to be replaced in one cable stayed bridge in Germany after only about 5 years of service. The results of rigerous inspection indicate that replacement of cables within expected life time of bridges must seriously be taken into consideration. The German Ministry of Transport considers provisions for cable replacement a must for all future cable stayed bridges.

3.2.5 Damping Devices

As mentioned above, the combination of dynamic effects and corrosion may present a serious problem. In addition, the aerodynamics of cables in service due to the combined actions of wind, dead load, live load and inter-related vibrations of cables, towers and superstructure is not yet fully understood. There are indications that only a certain combination of wind velocity, wind direction and mean stress in a cable cause vibrations where amplitudes in the order of several hundred millimeters have been observed. Normally even such amplitudes are relatively harmless from the stress point of view because of their seldom occurance. However, the psychological effect to road users should not be underestimated.

It stands to reason that damping devices should be installed wherever dynamic effects can be presumed to have adverse effects on the life expectancy of cables or where the behaviour of an existing bridge suggests that damping devices are installed as a rehabilitation measure. Fig. 8 and 9 present examples of possible solutions for the installation of damping devices as a rehabilitation measure. The spring-type dampers as shown on Fig. 9 were fixed to stiffened hand rails.

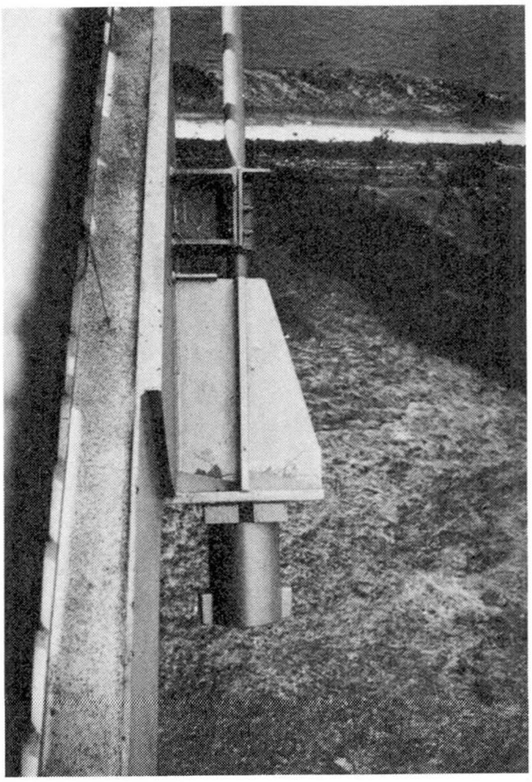


Fig. 8 Elastomeric Damper

Fig. 9 Spring Type Damper

3.3 Inspection

The development of an appropriate procedure and sequence for the inspection of all parts of a cable is important. In the past, the importance of a thorough and complete inspection of cables was often not acknowledged to the required extent by all designers of cable stayed bridges. Even an appropriate access to cable anchorages and cable sockets has not always been provided for in the design. Access to the free cable length between bridge deck and tower must also be possible and would normally require a special type of traveller. Fig. 10 shows a possible solution to this problem.

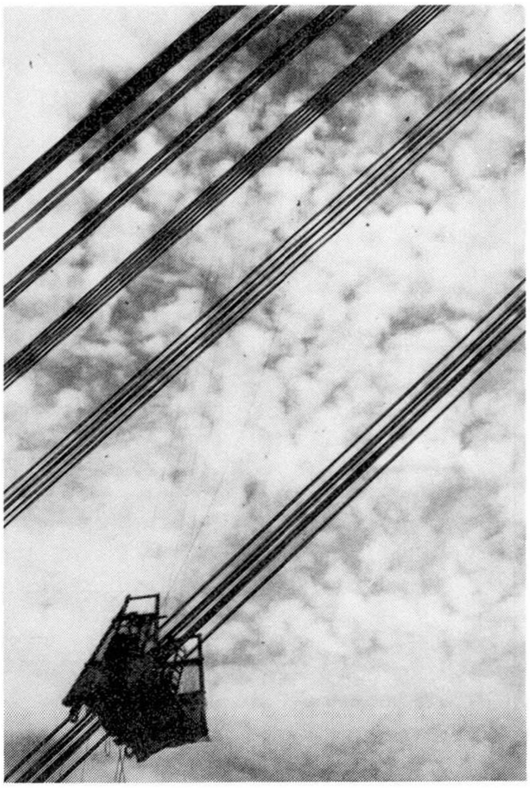


Fig. 10 Inspection of Cables

Reference is made to sub-chapter 2.2 regarding scale or intensity and frequency of inspection. Normally, the principal inspection will be the appropriate form of inspection, special inspection may be required in connection with unusual circumstances.

The general principal inspection will entail examination of the cable anchorages, cable sockets and condition of corrosion protection. An examination of the free cable length from the bridge deck, high quality binokulars will normally suffice, is strongly recommended. The interval between general principal inspections should not exeed three years.

The major principal inspection will be more intensive and require close examination of all parts. Free access to all parts is absolutely necessary. Special attention should be paid to possible wire breakage, cracks in sockets and anchorages including support elements, to a possible slip of wires, displaced shims, misalignement, movements and any kind of corrosion. The interval between major principal inspections will vary between 5 to a maximum of 7 years.

3.4 Corrosion Protection

In sub-chapter 2.4, it had already been stated that experience has clearly demonstrated only the best corrosion protection systems are capable to achieve the desired corrosion protection, so vital for the durability of all cable elements. Experience has also shown success can only be expected from a systematically built up corrosion protection. Since all exposed surfaces will require re-painting or even rehabilitation of the outside protection system during the expected service life of a cable stayed bridge, bridge designers are urgently called up to pay proper attention to a free access to all elements and to make allowances for all necessary provisions in order to accomplish high quality maintenance, repair and rehabilitation work to be done.

A systematically built-up corrosion protection is to start with the fabrication of cables in the shop. All wires should be galvanized firstly as an effective additional corrosion protection and secondly to provide for an additional mechanical protection during transport and erection but also during the following service life in a bridge. Mechanical damage caused by chippings, often applied in order to maintain a good skidding resistance of the bridge deck surfacing, are a well known occurance, and even willful damage can no longer be excluded. In addition to galvanizing a further corrosion protection of all interior wires is strongly recommended. The choice of materials should be dictated by the fact that an interior corrosion protection must last for the entire service life.

The wires of full-locked-spiral-cables are normally secured in the socket by a casting of a special zinc-alloy. The heat of the casting material may damage the interior corrosion protection in the cable ends near the socket. Special injection pipes are now being installed in the sockets in order to inject suitable material into the casting and into the cable ends. Tests have proved that the injected material extends into the undamaged zone of the cable.

"Main" cables in older cable stayed bridges, see also paragraph 3.2.3, are now being encased as a rehabilitation measure. The encasing is designed in such a way that the outside cables can be inspected in a major principal inspection. The encasing is then injected with suitable material in order to achieve a durable long lasting corrosion protection of the entire interior of the "main" cable.

A "standard" rehabilitation measure is now wrapping single cables in the bridge deck area as shown in Fig. 11 in order to provide additional protection in the brine spray zone and against any kind of mechanical damage. Best results can be expected if the wrapping is done before the paint is hardened.

In sum, corrosion protection should only make use of the best paint systems available in order to achieve an economical, long lasting durable protection of all corrosion prone elements against an increasing number of aggressive gases and soluble substances and taking into account mechanical damage that cannot be excluded in the vicinity of the bridge deck area.

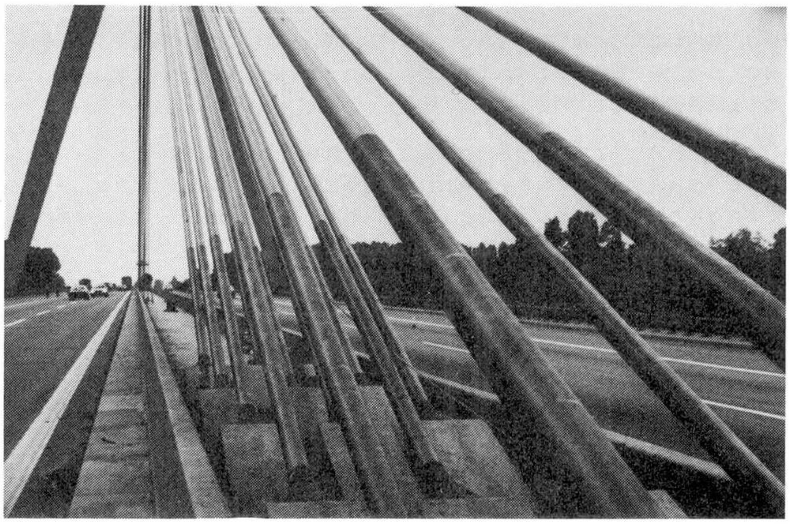


Fig. 11 Additional Wrapping in Splash-Zone

4. CONCLUSIONS AND RECOMMENDATIONS

4.1 Orthotropic Steel Deck

Orthotropic plates have a poor transverse load distribution. Most wheels of commercial traffic pass within a width which corresponds to the normal transverse stiffener spacing. These two facts combined indicate that orthotropic plates have heavily and less heavily loaded stiffener.

The development of an appropriate procedure and sequence for the inspection of orthotropic steel plates is important in order to ensure the degree of thoroughness and completeness in an inspection that is essential. Three forms of inspection can be recommended: the superficial inspection carried out quickly and frequently by highway maintenance personnel; the principal inspection, made by a trained inspector under the general supervision of a bridge engineer, falling into two categories referred to as general principal inspection and major principal inspection defined by frequency and intensity; the special inspection, normally made in connection with unusual circumstances.

Cracks have been found especially in the stiffener to cross-beam welds. Some cases are known where rusting took place in closed stiffeners that are nominally sealed by welding.

Temperature gradients, inherent to orthotropic plates, cause condensation which in turn causes rusting on the underside of stiffeners. It is recommended to use only the best paint systems available. Since condensation is a permanent occurance, it is recommended that intervals between application of paint layers be kept as short as possible.

Resurfacing is nowadays normally machine laid in order to economize on the laying operation and to minimise delays to traffic. Thermal effects on the structure due to machine laying can no longer be neglected. It is recommended that a total reassessment of the structure is made before resurfacing in order to avoid possible damage to the structure. Information is presented in sub-chapters 2.6 and 2.7 as an aid to diagnose thermal effects.

4.2 Cable Stayed Bridges

A tensile strength not exeeding 1600 N/mm² is believed to be a sound compromise between required tensile strength and sensivity to corrosion and stress corrosion.

Single cable stays are recommended. They allow free access to inspection and necessary maintenance work. Since cables may have to be replaced during the service life of a cable stayed bridge, the bridge should be so designed as to allow for cable replacement in a bridge in service.

Damping devices may be required in order to counteract aerodynamic effects and to relieve unfavourable stress situations near the anchorages. Installation of damping devices can in most cases be done as a rehabilitation measure in existing bridges.

Proper inspection of all cable elements is essential. Free access is a presupposition for the required quality of inspection. Normally, the principal inspection will be the appropriate form of inspection.

A thorough and complete corrosion protection is vital for the expected durability of all cable elements. Corrosion protection systems should be built-up systematically and should start with the fabrication of cables in the shop. Corrosion protection should only make use of the best paint systems available. As a rehabilitation measure additional protective wrapping in the vicinity of the bridge deck area is recommended.

REFERENCES

1. Fraunhofer-Institut für Betriebsfestigkeit, Darmstadt. Anhang zum Schlußbericht: Messung unter definierter Belastung eines zweiachsigen Fahrzeugs an einer Schrägseilbrücke, Februar 1982.

2. LEBEK, D.E., KNABENSCHUH, H., Untersuchungen über Temperaturbelastungen von Stahlbrücken mit orthotroper Fahrbahnplatte und ihrer Korrosionsschutzsysteme beim Einbau von bituminösen Fahrbahnbelägen, Bundesanstalt für Strassenwesen, Juni 1976.

3. LEONHARD UND ANDRÄ, Gemeinschaft beratender Ingenieure, Beanspruchung von Stahlbrücken beim Einbau bituminöser Fahrbahnbeläge aufgrund gemessener Temperaturverteilungen, Bundesanstalt für Strassenwesen, Dezember 1980.

4. BUNDESMINISTERIUM FÜR VERKEHR, Forschungsvorhaben FA 15.079R78H, Betriebsfestigkeitsuntersuchung für seilverspannte Brücken,Teil B, Betriebsfestigkeitsuntersuchungen für Brückenseile, Ingenieurbüro Grassl, Dezember 1981.

Rehabilitation and Repair of Bridges

Réparation des ponts

Sanierung und Instandstellung von Brücken

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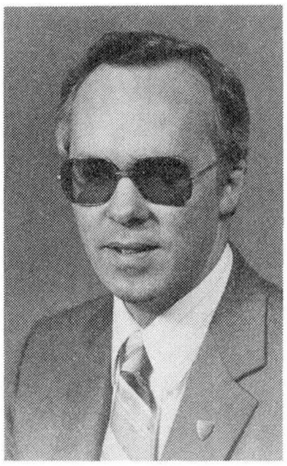

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SUMMARY
The correct level of maintenance for a bridge structure is a function of its need and the benefit it provides. Structures whose justification for existence is marginal or non-existent or that should not have been built in the first place are not worth maintaining beyond minimum safety levels for their actual use. This paper will illustrate these principles together with a number of repair schemes.

RESUME
Le niveau optimal d'entretien d'un pont dépend de l'utilité de celui-ci et de l'amélioration physique qu'on peut y apporter. Les structures dont l'emplacement et l'existence ne peuvent être justifiés d'un point de vue économique doivent être entretenues au strict minimum qu'exige la sécurité. L'article expliquant ces grands principes est illustré par quelques modèles de réparation.

ZUSAMMENFASSUNG
Der korrekte Stand des Unterhalts für eine Brücke ist eine Funktion ihrer Notwendigkeit und ihres Nutzens. Bauwerke deren Existenzberechtigung marginal oder nicht existent ist oder welche nicht hätten gebaut werden sollen, sollten mit einem absoluten Minimum an Aufwand für die geforderte Sicherheit unterhalten werden. Der Beitrag beschreibt anhand einiger Instandstellungsmodellen diese Prinzipien.

We should view the Maintenance Repair and Rehabilitation of Bridges as a circular process. The first step covered in session one is the "eyes" or inspection phase. The second is the evaluation phase covered in session 2. The third phase is the action phase covered in this session and part of the next. This must be followed by an inspection to ensure the desired results were achieved.

Part of the evaluation and action phases is a determination of how bad the situation really is and what action must be taken. A major ingredient in this is the available or necessary funding which will be more fully covered in the last session. A major problem in this area is that of choosing a proper interest rate for economic decision making. It does not seem right to use interest rates of 20% in deciding whether to paint or not while not including inflation. In profit making companies it does not seem right to declare a profit while due to lack of sufficient maintenance funds the assets of the company are allowed to deteriorate.

If we have been consuming our capital through inadequate maintenance then whatever theory that proports to justify the practice is wrong and not the reality.

Other papers in this session and the results of NCHRP (National Cooperative Highway Research Program) projects 12-20, (1,2) 12-21 (3), 12-15 (4) and other similar studies discuss various repair and rehabilitation schemes. The purpose of this presentation is to serve as an introduction to these and to remind all of the objective of these repair and rehabililtation schemes.

The correct level of maintenance for a bridge structure is a function of its need and the benefit it provides. For all structures a minimum level of safety consistent with the use to which it is actually put is essential. Nevertheless, without being foolhardy, it is sometimes amazing how far materials can be pushed. There are numerous examples of structure that do not meet current standards that are nevertheless quite adequate.

There are many structures that don't meet current geometric standards. Either their clearance or alignment is inadequate.

The solution shown in figure 1 is a lot more economical than building a new bridge. The construction of a new bridge can wait until the traffic warrants it, since the occasional high truck can get across by using a short detour.

It will be a long time before many structures need to be rebuilt to a new alignment as they are infrequently used and are located in areas where excessive speed is not possible.

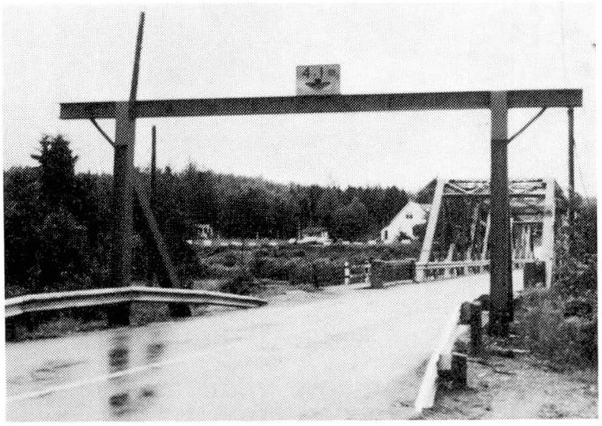


Figure 1 Height Protection

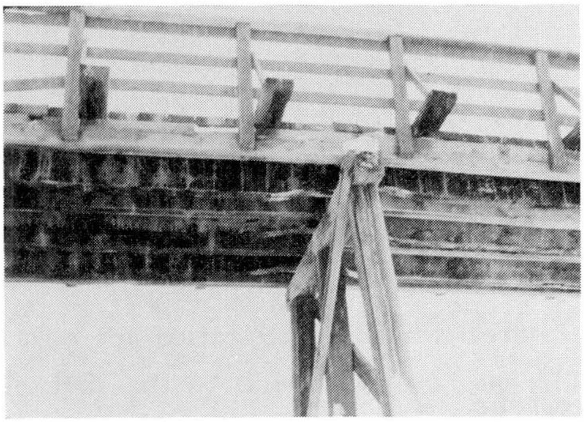


Figure 2 Crushing Timber

The structure shown in Figure 2 shows crushing of the outside stringers and floor beams which are wooden beams. Nevertheless, the center of the structure is used to get one farm tractor across daily. By blocking off the edges, repairs or replacement are avoided.

In the case of another structure, which again serves only one user, as soon as the structure can no longer safely handle his vehicle, it will be closed and a small level crossing will be built at a fraction of the cost. Railway traffic in the area is expected to remain light for some time, so this will not create a safety hazard.

The structure shown in Figure 3 has been carrying mainline railway traffic for three years with the cracks shown. The cracks are monitered annually and if they remain dormant the span won't be replaced for at least another 5 years, at which time they will be replaced for other reasons.

Sometimes the no action solution is the most economical.

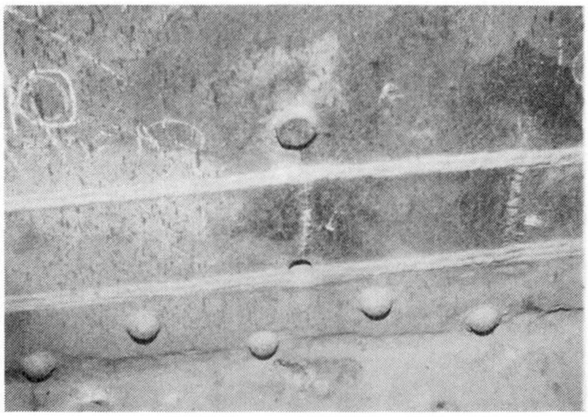


Figure 3 Crack in Girder

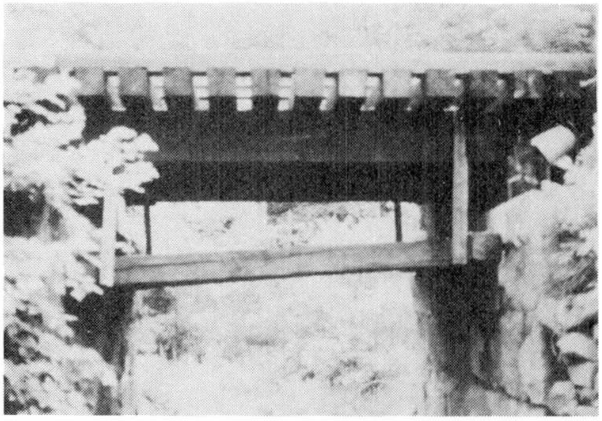


Figure 4 Temporary Support

Structures whose justification for existance is marginal or non-existent or that should not have been built in the first place are not worth maintaining beyond minimum safety levels for their actual use. In fact, if the economic hardship to the users is not too great, breakdown maintenance or closure might be the most viable solution.

Figure 4 shows a structure which has been strengthened sufficiently to last one more winter at which time the railway line is to be closed. Traffic will then be re-routed.

There are other structures which will be kept in service as long as possible with minimum maintenance, as they serve lines which are only profitable as long as maintenance expenditures are kept to a minimum. In one case, because there are 20 structures in similar condition on the line, it would cease to be profitable if full rehabilitation were considered. To do full scale repairs would force us to raise our rates sufficiently to loose the traffic to local trucking companies. This line is a perfect example of a marginal investment which will be kept open until it is no longer safe to do so with minimum maintenance expenditure. In cases where the service is in the public interest, governing authorities pick up all or part of the maintenance costs.

Figure 5 shows a structure over which operations have ceased. This structure became unsafe recently. To rehabilitate it to new condition would cost $7. million dollars, to fix it to last until the next spring ice movement would cost over $400,000. The saving to the railway by using the bridge was of the order of $40,000 before considering any maintenance expenses. I must say that the initial damage to the structure was caused by the action of others who changed the ice conditions. Should the structure be rehabilitated, they will probably be held liable. Nevertheless, we have found another way to adequately serve our customers and are not throwing away money which can be better spent elsewhere.

Figure 5 Moved by ice

One must be very careful with economics. One of our lines, which carriers very little traffic, generates a large profit since the 400 or so cars per year from the line travel 3/4 of our country afterwards generating far more revenue than the maintenance of the line requires. One must always check the theory with reality, especially economic theory.

As resources become scarce, we must be more selective in allocating resources to maintenance. Those structures that are worth having should be maintained to a high standard because it is generally more economical to preserve the asset than to permit it to deteriorate.

The maintenance painting of our large cantilever bridge at Quebec has been kept in our budget in spite of a 50% cut back in funds as it is a worthwhile investment and must be maintained. As long as painting is done regularly, costly sandblasting is not required.

In other areas of the country, the salt spray is so severe that re-painting without sandblasting would be a waste of money.

There are (5) studies which show that preventive maintenance is the least expensive way of maintaining a structure that has full economic justification for existence. The structure shown in Figure 6, built in 1935, is so well maintained that you could almost eat off it, and it has proved over the years to be very inexpensive to maintain. Constrast this with the member shown in Figure 7, built the same year, whose painting has been neglected.

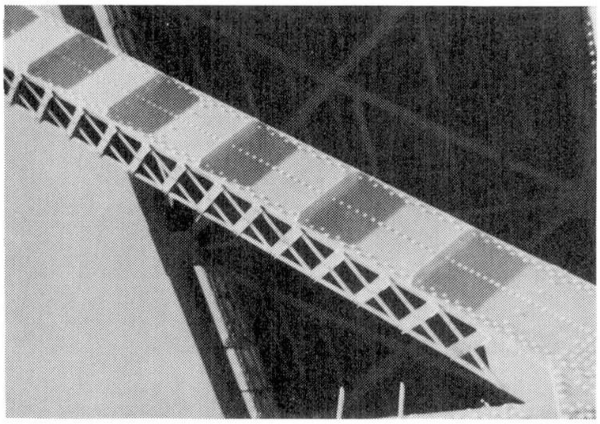


Figure 6 Well painted
(Photo by C. Seim)

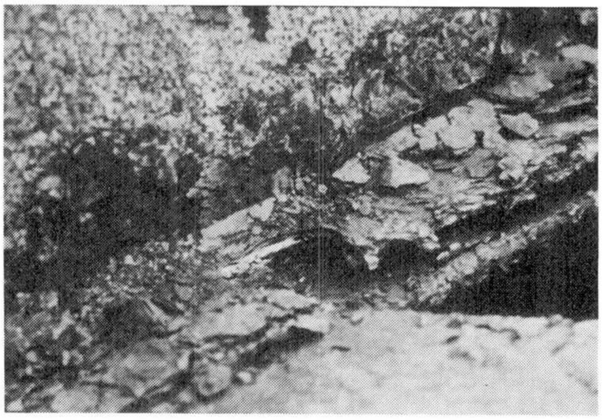


Figure 7 Not painted

In our homes, we all know that it is cheaper to repaint a window frame than to leave it until the wood and perhaps the insulation in the wall has to be replaced, or to replace a few roof tiles before the water and ice enter the walls and cause them to buldge. Yet, when short of funds, the temptation to say that no bridge has ever fallen down because it missed one year's paint exists. The consequences of deferred maintenance come later with a vengeance when an industry or nation can no longer compete because it cannot afford the capital investment to rebuild completely.

Just imagine if all the structure on a railway line were permitted to deteriorate to the level shown in figure 7. Eventually it would be necessary to generate sufficient funds to completely rebuild the whole line. We in North America have lived in a throw away society and this has been applied to most of our industrial plant and many of our bridges, both public and private. Our observations of the consequences show us that this has not been a wise policy.

Nevertheless, if a structure cannot be justified, it is hard, I say impossible, to justify its maintenance. The same is true if only half a structure is justified then maybe only half maintenance is justified?

If safety becomes a problem the alternative of closing the bridge must be considered. Pouring money into structures that are not needed or that can do the job as is, is a terrible waste.

Structurally weak bridges should be maintained to a higher than normal level consistent with their use. The bridge shown in Figure 8, built in 1904 is one of the heaviest travelled and yet weakiest of our main line bridges (6, 7). Because of extremely good preventative maintenance the structure serves us well.

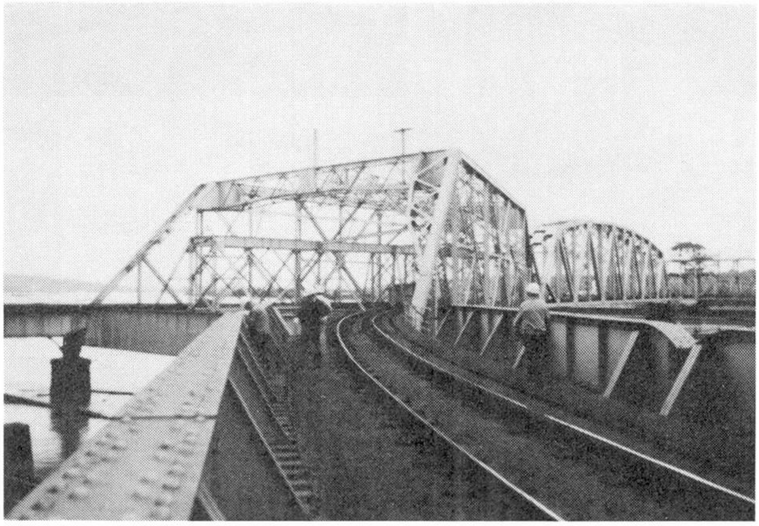


Figure 8 Well maintained

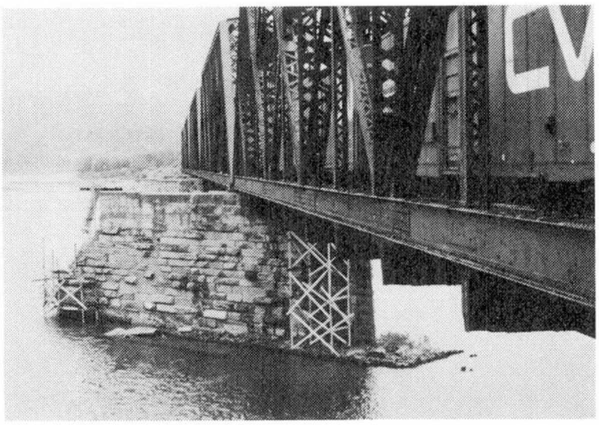
Having questioned whether the maintenance, repair or rehabilitation is in fact appropriate, I would like to illustrate several repair schemes, not covered by others in this session.

Since more than half of all bridge failures are caused by water through flooding, undermining or debris, we might as well start there.

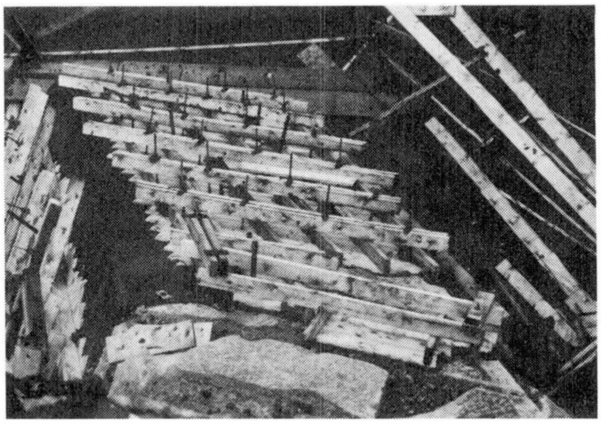
The simple preventative step of keeping stone masonry properly pointed will usually avoid subsequent more costly repairs caused by movement of stones. Once a structure is permitted to loose a stone or two, it becomes costly and technically difficult to repair (Figure 9).

Among the most common techniques are placing bags filled with cement or concrete as a form and either placing tremi concrete or placing aggregate and pumping grout. Larger jobs require extensive forming and reinforcing adequately dowled to the existing structure.

The underpinning, straightening or strengthening of a pier can be quite an undertaking.



A - Overview



B - Close-up

Figure 9 Repair to pier

Similar techniques can be applied to concrete piers and abutments.

The maintenance of soil-steel structures can be quite a task if they were improperly installed. Mechanical straightening, followed by qrouting the surrounding soil will usually solve the problem.

The second most prevalent cause of bridge failures is corrosion. Those who maintain reinforced and prestressed concrete structures are learning how difficult it is to adequately handle this problem. Adequate waterproofing protection and confinement of steel is technically and economically quite a challenge as some of the other papers will show. Preventative maintenance is easier said than done.

Because of the disruptions to traffic in replacing defective or non-existent waterproofing, these protects are often delayed until it is too late. Complete deck replacement may be the only feasible solution.

In steel structures, it is very tempting to delay preventative maintenance painting when funds are tight. After all, no structure has ever fallen down because of a one year delay in painting. Those who have watched automobiles rust away know that an adequate paint job at the right time could have saved the vehicle. The same is true of a bridge. A one year delay can allow corrosion to go far enough that is is much more costly and in some environments not possible to stop. In some locations successfully applying a coat of paint is a technical achievement.

When the web of a girder has reached a stage where it can no longer adequately carry the load, then a bolted replacement web Figure 10, is possible. If the corrosion is confined to the web near the stiffener, the stiffener can be removed and a small plate added to the web before replacing the stiffener.

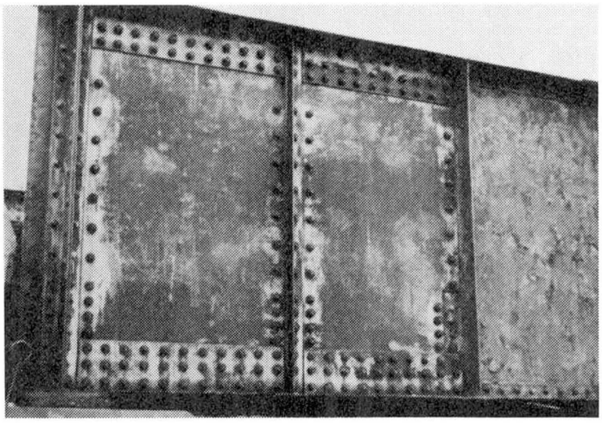


Figure 10 Replacement Web

I must warn against welding these replacements as weld terminations have very low fatigue strength. Cracks initiated in patch plate welds can go into the original web causing a potentially dangerous situation. Welding to a non-welded structure can destroy the inherent component redundancy in the structure (8). In a bolted or riveted girder, a crack in one plate or angle will not propogate to the rest of the member. Join these with a weld and the crack can propogate.

I would recommend the repair shown in Figure 11 to replace the section lost due to corrosion of the web just above the bottom flange. This is a much better solution than the one shown in Figure 3 where the welds cracked.

In the case of yielded, buckled or cracked members, it is possible to splice around the failed part in such a way that all load can be carried by the splice. The repair shown in Figure 12 shows a splice around members that cracked after being exposed to a very severe fire. The extent of locked in residual stresses was of such concern that a significant part of the member was spliced.

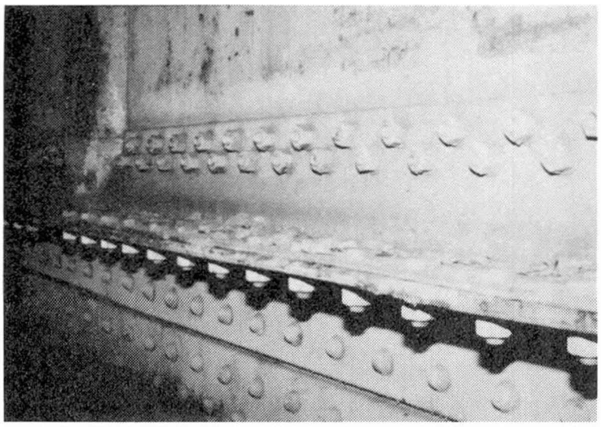


Figure 11 Repair to bottom of web

Figure 12 Member spliced

Care must be taken in the connection of all repair and rehabilitation schemes. The strengthening of a deck truss shown in Figure 13 was not very successful. The staggered welds eventually broke.

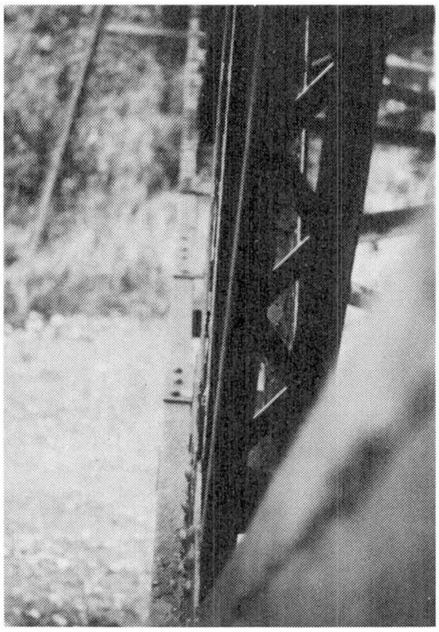


Figure 13 Broken welds

In trusses, when the pins or pinplates wear they can be replaced by larger pins after reaming to make the hole larger. If this is not possible, then a pin joint can sometimes be replaced by gusset plates. In order to minimize future fatigue problems, it is important to be very careful about compatible defections to ensure that the replacement gusset plates equally share all loads (Figure 14). If one plate carries no load, then the doubled stress range in the other could lead to unexpected fatigue problems (6,7).

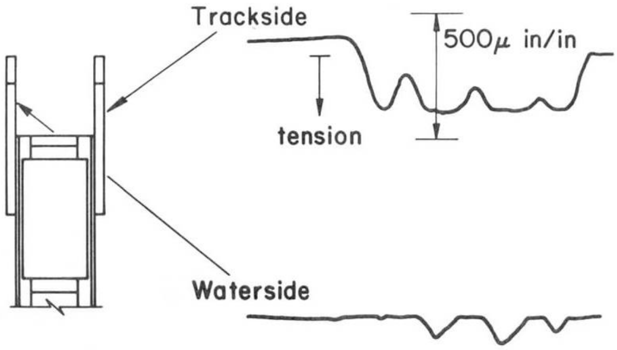


Figure 14 Unequal loading

In older mild steel structures, a differential strain of one thousand of an inch can cause the relief of stress approaching the yield point. Rehabilitation schemes must be planned so that the replacement parts take the load in the way expected. This is not something that can be left to skilled tradesmen. It must be engineered.

Workmanship is just as important when making permanent repairs as in new construction. For example, copes must have proper radii and be ground smooth otherwise cracking may result.

If stiffening is required during jacking, it must be specified or the results could be a bent flange. A timber stiffener jambed betweem the top and bottom flange can often suffice.

Simple preventative measures such as cutting bushes to reduce moisture can retard the corrosion of the base of viaduct towers. Keeping structures clean can allow proper venting which greatly reduces corrosion rates.

During repairs it is a good time to get at the cause and not just the symptom. Mechanically replacing or repairing existing details can be a terrible waste.

In setting up maintenance organizations we must not fall into the trap of replacing in kind but must always check to see if the repair or retrofit should be done, and if so is a repair in kind the most appropriate.

In one of our concrete box girders, water was trapped inside because the small drain got plugged. When the water froze it burst the top slab and heaved the track. A proper retrofit in this case was to make the drain large enough that it could not be so easily blocked.

The crack shown in Figure 15 started where the gusset plate to vertical stiffener weld terminates (10). Many of these cracks were not detected until the crack front had reached the stiffener to web weld. In order to stop the crack from proceeding up or down this weld, holes were drilled. These holes, which were drilled from both sides of the interior stiffener and then from the outside, were quite difficult to do. If the crack is not stopped then a more costly retrofit would be necessary. A permanent repair is shown in Figure 16 (9).

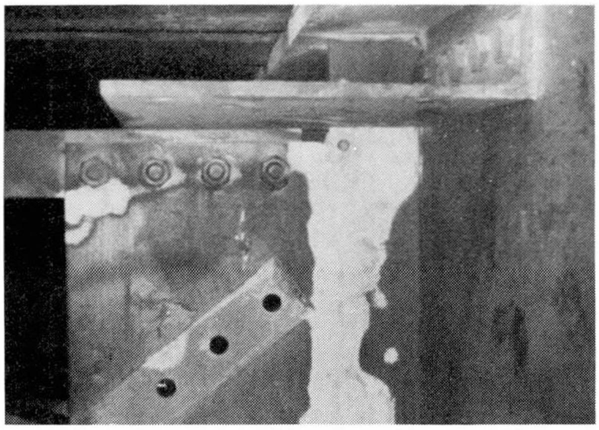


Figure 15 Crack at sharp notch

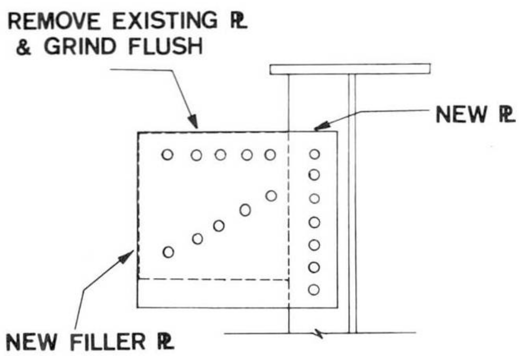


Figure 16 Typical repair

The structure shown in Figure 17 has cracked and been repaired once every ten years when in fact the problem is that there is no where for breaking forces to be dissipated.

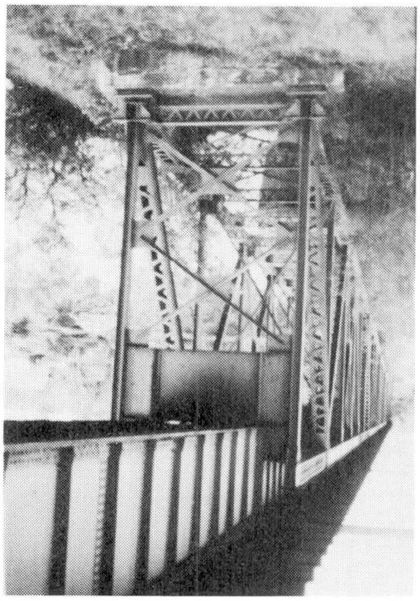


Figure 17 End of deck truss

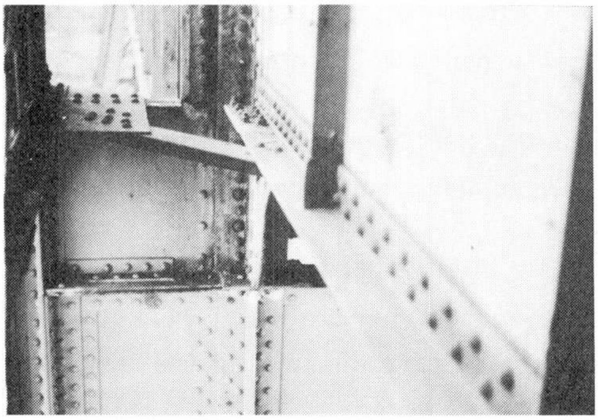


Figure 18 Bracing leads nowhere

The traction bracing ends at the top of the deck truss (Figure 18). Since the end post is relatively flexible, the load is forced to try and go through the deck plate girder. The connection is not designed for this force and movement, so it cracks. Changing the location of traction braking in order to get it directly to the bearings of the truss would permanently solve the problem.

In some older structures, the material was not placed in the most effective position. Deterioration can be permitted in the upper shelf angles of a compound tension flange if calculations confirm the strength of the flange.

Repairs require some thought since structures do not always behave as assumed. Consider the bottom chord in the first panel of a deck truss. In one case although tension was expected the member was in compression. Because the bearings were completely frozen or unable to move, the truss acted like an arch or prestressed truss. Strengthening with say loose cables would not have been effective.

In the event of uncertainty, strain measurements may be necessary.

The next most common cause of bridge failure is fatigue or wear related. Great progress has been made in our understanding of this area in the past decade.

We must be careful to ensure that our repairs will stand the test of time and not be worse than no repair at all. Repairing minor corrosion with welded patch plates with poor fatigue strength is not wise. Repairs should prolong and not shorten the life of the structure.

One technique, we have used, is to peen the ends of shallow cracked welds (Figure 19) to prolong their life. In other cases, replacing poorly installed rivets with high-strength bolts can prolong the life of a connection.

Figure 19 Peening welds

As we come out of the throw away society, there will be plenty of challenges to find successful repair and retrofit methods to difficult cases as the other papers illustrate. In many cases the rehabilitation of a bridge is an extremely difficult technical task limited by many constraints. Nevertheless, the most difficult task will be to convince authorities that the most cost effective route is through a very high level of preventative maintenance for those structures that are worth maintaining.

Maintaining structures to ideal standards, given adequate funds, is relatively easy. The challenge for our profession is that maintenance must not only be safe, cost-effective and environmentally sound, but must also be resource-efficient, technologically appropriate, and socially necessary.

REFERENCES

1. "Bridges on Secondary Highways and Local Roads-Rehabilitation and Replacement." NCHRP Report 222 (1980), Transportation Research Board, Washington, D.C.

2. "Rehabilitation and Replacement of Bridges on Secondary Highways and Local Roads." NCHRP Report 243 (1981), Transportation Research Board, Washington, D.C.

3. Shanafelt, G.O., and Horn, W.B., "Damage Evaluation and Repair Methods for Prestressed Concrete Bridge Members." NCHRP Report 226 (1980), Transportation Research Board, Washington, D.C.

4. Fisher, J.W., Hausamaun, H., Sullivan, M.D., and Pense, A.W., "Detection and Repair of Fatigue Damage in Welded Highway Bridges." NCHRP Report 206 (1979), Transportation Research Board, Washington, D.C.

5. Fitzpatrick, M.W., Law, D.A., Dixon, W.C., "Deterioration of New York State Highway Structures", T.R.B. Record 800 (1980), Transportation Research Board, Washington, D.C.

6. Sweeney, R.A.P., "Load Spectrum for Fraser River Bridge at New Westminster, B.C.," Proceedings American Railway Engineering Association Vol. 77, Bulletin 658, June-July 1976.

7. Fisher, J.W., and Daniels, J.H., "An Investigation of the Estimated Fatigue Damage in Members of the 380 ft. Main Span, Fraser River Bridge," Proceedings American Railway Engineering Association, Vol. 77, Bulletin 658, June-July 1976.

8. Sweeney, R.A.P., "Importance of Redundancy in Bridge Fracture Control" T.R.B. Record 711 (1979), Transportation Research Board, Washington, D.C.

9. Sweeney, R.A.P., "Some Examples of Detection and Repair of Fatigue Damage in Railway Bridge Members," T.R.B. Record 676 (1978), Transportation Research Borad, Washington, D.C.

10. Sweeney, R.A.P., "Some Remarks on the Service Behaviour of Steel Railway Bridges," IABSE Symposium - Bridges, Zurich 1979.

Seismic Retrofitting of Highway Bridges

Renforcement de ponts routiers en vue de séismes

Verstärkung von Strassenbrücken im Hinblick auf Erdbeben

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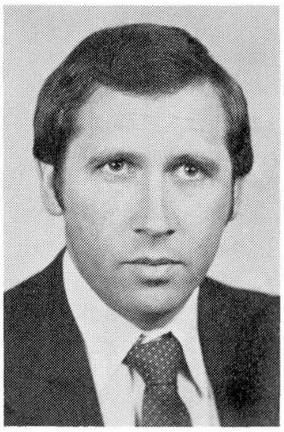

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SUMMARY
The paper presents a progress report on the Applied Technology Council ATC-6-2 project to develop guidelines for the seismic retrofitting of existing highway bridges. The guidelines will be applicable for all regions of the United States and for bridges of conventional steel and concrete girder and box girder construction with spans not exceding 180 m. The guidelines will provide a preliminary screening procedure to develop a priority rating for bridges in a particular region. Once a decision has been made to retrofit a particular bridge the guidelines provide a detailed evaluation procedure to determine seismic demand/capacity rations for all seismically vulnerable components of a bridge. The procedure can also be used to evaluate the effectiveness of any retrofit scheme contemplated by the engineer.

RESUME
L'article présente un rapport intermédiaire du projet de l',,Applied Technology Council'' sur les directives pour le renforcement des ponts routiers en vue de séismes. Les directives sont applicables dans toutes les régions des Etats-Unis, pour des ponts métalliques et en béton ainsi que pour des ponts caisson dont les portées ne dépassent pas 180 m. Les directives proposent une procédure préliminaire permettant d'établir des priorités parmi les ponts à renforcer dans une région particulière. Une fois la décision prise de renforcer un pont, les directives présentent une procédure d'évaluation détaillée afin de déterminer les rapports résistance au séisme-charge pour tous les éléments du pont vulnérables au séisme. La procédure pourra être également utilisée pour évaluer la valeur de tout projet de renforcement.

ZUSAMMENFASSUNG
Ein Zwischenbericht des Projektes des ,,Applied Technology Council'' über die Richtlinien zur Verstärkung von Strassenbrücken gegenüber Erdbeben wird vorgestellt. Die Richtlinien sollen auf dem ganzen Gebiet der Vereinigten Staaten anwendbar sein, für Stahl- und Betonbrücken, sowie Hohlkastenbrücken, deren Spannweite 180 m nicht übersteigt. Die Richtlinien schlagen ein Vorverfahren vor, welches Prioritäten unter den zu verstärkenden Brücken in einem gegebenen Gebiet erfassen lässt. Nach dem Entscheid, eine Brücke zu verstärken, erlauben die Richtlinien, mittels einem detaillierten Schätzungsverfahren das Verhältnis Erdbebenlasten zu Tragfähigkeiten für alle erdbebengefährdeten Brückenbauteile zu bestimmen. Das Verfahren kann auch für die Bewertung der Wirksamkeit eines beliebigen Verstärkungsprojektes benützt werden.

1. INTRODUCTION

The collapse of a highway bridge during an earthquake will in many cases sever vital transportation routes at a time when they are most needed to provide emergency services to or facilitate evacuation from a stricken area. The loss of the bridge as a transportation link may potentially result in a greater loss of life than the immediate effects of collapse.

The San Fernando Earthquake of 1971 taught engineers a great deal about the seismic resistance of bridge structures and resulted in the development of improved provisions for the design of new highway bridges.[1,2] This earthquake also demonstrated the potential inadequacy of past design procedures in providing seismically resistant bridges. Since most existing bridges in service today were designed using pre-1971 design procedures, it follows that many of the nations highway bridges in seismically active areas may have insufficient strength to resist seismic loading.

The problem of the seismic safety of our existing highway bridges is widespread and of sufficient magnitude to warrant national attention. Although bridge failures due to earthquakes have been confined to Alaska and California, many of these failures occurred at relatively low levels of ground motion. Seismologists have estimated that 37 of the 50 states and Puerto Rico have the potential for ground motion of a magnitude greater than or equal to that which has resulted in serious bridge damage in past earthquakes. Comparatively high levels of ground shaking can be expected in fifteen of these states and Puerto Rico. The potential for bridge failure in many states may also be aggravated by a previous lack of emphasis on seismic design. Certain structural details, especially at bearings, have proved extremely vulnerable to damage in past earthquakes.

To deal with this seismic safety problem it is necessary that an effort be made to identify deficient bridges, evaluate the risk and consequences of serious damage, and initiate a program to mitigate the risk of seismic failure. One method for dealing with the risk of failure, first initiated by the California Department of Transportation following the San Fernando Earthquake, is to strengthen seismically deficient bridges. This process is commonly referred to as seismic retrofitting.

Applied Technology Council is currently engaged in a project funded by the Federal Highway Administration to develop comprehensive guidelines for seismic retrofitting of bridges. These guidelines will include methods of identifying seismically vulnerable bridges, procedures for evaluating the existing seismic capacity, and methods and design details to improve the seismic resistance of bridges. This paper describes the background to the project, and the progress to date toward the development of the guidelines.

2. BACKGROUND

Applied Technology Council has recently completed a project (ATC-6) to develop "Seismic Design Guidelines for Highway Bridges".[1] These guidelines are intended for use in the seismic design of new structures. They are the result of a concerted effort by the members of an ATC Project Engineering Panel (PEP). The PEP was composed of representatives from bridge design firms, state highway departments, universities, the Federal Highway Administration and Applied Technology Council.

The current project, which is directed toward seismic retrofitting of existing bridges, is an extension of the effort to develop guidelines for the seismic design of new structures. Many of the principles being considered for use in retrofitting were first developed as part of the seismic design guidelines. A review of these principles follows.

2.1 Seismic Design Guidelines for Highway New Bridges

The seismic design guidelines for new bridges were developed for national use, and therefore contain provisions for considering the variable levels of expected seismic activity in the United States. This is primarily done through the use of an acceleration coefficient, A, which is the effective horizontal ground acceleration at the bridge site. An earthquake producing this acceleration has a ten percent chance of being equaled or exceeded at a given site within a 50 year period. A contour map of acceleration coefficients is shown in Figure 1. This map was developed by a team of seismologists using both historical and geological data.

The guidelines also consider the importance of the structure in social/survival and security/defense terms. Essential bridges are assigned a high importance signified by an Importance Classification I. All other bridges are placed in Importance Classification II. The Importance Classification is used along with the acceleration coefficient to assign a bridge to one of four seismic performance categories, A through D, as shown in Table 1. The analysis and design requirements vary depending on the seismic performance category.

TABLE 1: SEISMIC PERFORMANCE CATEGORY

Acceleration Coefficient	Importance Classification I	Importance Classification II
$A \leq .09$	A	A
$.09 < A \leq .19$	B	B
$.19 < A \leq .29$	C	C
$.29 < A$	D	C

The seismic design guidelines utilize one of two elastic response spectrum analysis procedures to determine the seismic displacements and elastic member forces due to the design earthquake in each of two perpendicular horizontal directions. Provisions are given for combining forces resulting from earthquakes in the two horizontal directions to account for directional uncertainty of the earthquake. The response spectra to be used vary based on acceleration coefficient and the soil profile at the bridge site.

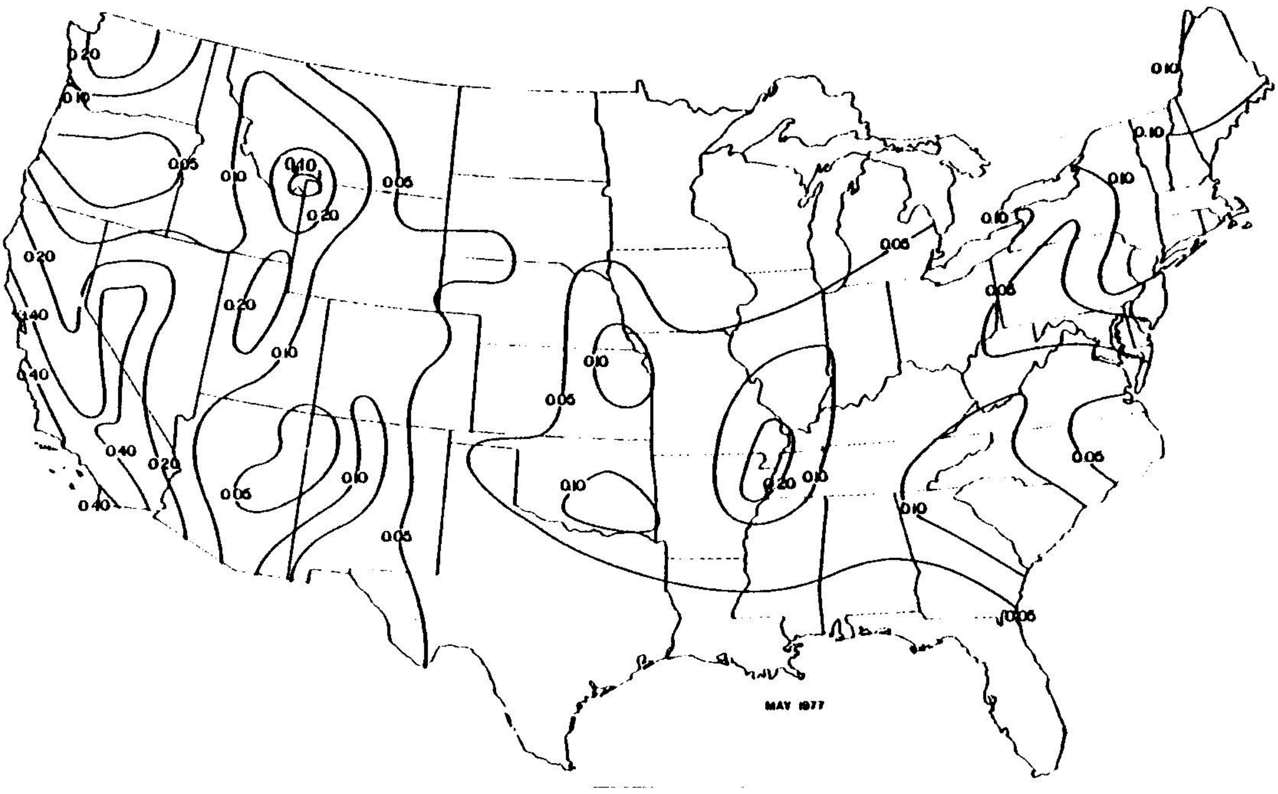

Fig. 1 Acceleration coefficient - Continental United States

The elastic member forces obtained from the analysis are divided by response modification factors to obtain component design forces. Use of a response modification factor greater than one implies the acceptance of yielding in the member. Use of factors less than one are used for non-ductile components that may be subjected to higher forces due to yielding elsewhere in the structure. The guidelines also allow for a reduction in certain design forces when it can be shown that column yielding will limit these forces to certain maximum values.

Elastic displacements form a lower bound on the expected structure displacements. To account for larger relative displacements at expansion joints, the guidelines specify minimum support lengths. These support lengths are intended to account for the overall inelastic response of the bridge structure, possible independent movement of different parts of the substructure, and out-of-phase rotation of abutments and columns resulting from traveling surface wave motions.

Special design requirements are provided for foundations and abutments, structural steel, and reinforced concrete. Particular attention is given to the reinforcement details of concrete bridge columns. These details are directed toward providing greater ductility in columns which are assumed to undergo inelastic yielding.

3. RETROFIT GUIDELINES

A Project Engineering Panel is also being used to develop the seismic retrofit guidelines for highway bridges. The panel for this project is composed of consulting engineers, academicians, state highway engineers, and representatives from the Federal Highway Administration and Applied Technology Council. The project is expected to be completed in early 1983.

The retrofit guidelines will recommend procedures for evaluating and upgrading the seismic resistance of existing highway bridges. Methods of evaluation will assist engineers in identifying and assessing bridges which could be hazardous to life safety during earthquakes. Evaluation results may be used with engineering judgement to decide if, how and to what degree a bridge should be retrofitted.

Methods of retrofitting various vulnerable bridge components will also be presented in the guidelines. Since seismic retrofitting is a relatively new concept, only a few retrofitting schemes have been tried in practice. At present, seismic retrofitting is an art requiring considerable engineering judgement. Although the guidelines will present accepted retrofitting techniques, they are not intended to restrict innovative designs.

The guidelines will not prescribe rigid requirements dictating when and how every bridge is to be strengthened. Retrofitting decisions depend on several factors, many of which are economic and are outside the realm of engineering. The guidelines are intended to provide a guide for rationally assessing the engineering factors involved.

The retofit process involves identification of the bridges which pose the greatest threat to life safety due to earthquakes; a procedure for the detailed evaluation of individual bridges so identified; determination of the need for retrofitting; identification of appropriate retrofit measures; an economic assessment of the benefits of retrofitting; and a decision to retrofit or not to retrofit. This process is depicted in the flow chart shown in Figure 2. The guidelines are intended for use throughout this retrofitting process. Specifically, the guidelines will provide:

- A preliminary screening process for determining which bridges present the greatest hazard due to earthquakes
- A methodology for quantitatively evaluating the seismic capacity of an existing bridge
- Retrofit measures that can be used to increase the seismic resistance of bridges

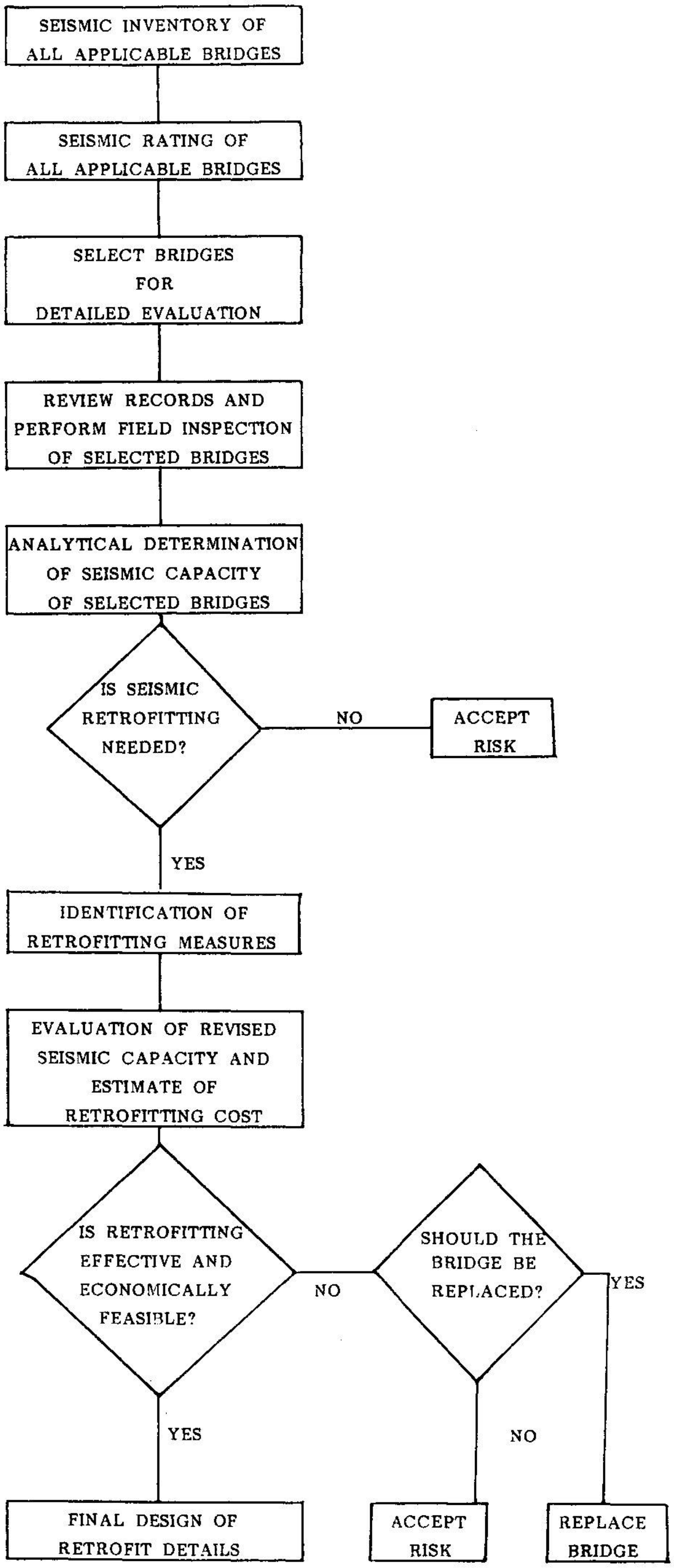

Fig. 2 Seismic retrofitting process for bridges

Each of these features of the guidelines will be discussed in greater detail in the sections that follow.

3.1 Preliminary Screening Process

The first problem facing an engineer contemplating a seismic retrofitting program is to determine which bridges should be retrofitted first. If a large number of bridges are being considered, this determination must be accomplished with a minimum of effort for each bridge. The retrofit guidelines will establish a framework for addressing this problem through the use of a seismic rating system. This system will help the engineer establish a priority in which bridges should be investigated for retrofitting by a more detailed evaluation.

The seismic rating system will be subjective by design. This will allow the engineer to consider regional and jurisdictional needs in assigning priorities.

To enhance consistancy it is recommended that rating of all bridges in one geographical area be performed by the same personnel.

The seismic rating of a bridge will consider structural vulnerability, seismicity and structure importance. Each of these three areas will be assigned an independent rating, weight and score. The scores will be added to arrive at an overall seismic rating according to the following procedure:

1.	Vulnerability Rating (rating 0 to 10) x weight	=	score
2.	Seismicity Rating (rating 0 to 10) x weight	=	score
3.	Importance Rating (rating 0 to 10) x weight	=	score
	Seismic Rating (100 maximum)	=	Total Score

Although the ratings in each of the three areas will be established by procedures described in the guidelines, the weights to be used, which must total 10, will be left to the discretion of the engineer. This will allow the engineer the flexibility to emphasize the seismic aspects most important to him. For example, in an area with uniform seismicity, the seismicity rating may be deemphasized by using a low weight. This will minimize the effect of seismicity and emphasize vulnerability and importance in the overall rating. Conversely, in an area of variable seismicity, it will be desirable to place more emphasis on seismicity. In this case a greater weight should be applied to the seismicity rating.

Other more scientific methods for combining the affects of vulnerability, seismicity and importance were considered. The objective of the rating system was limited, however, to providing the engineer with a framework for systematically considering the three most important engineering aspects of the retrofitting problem. In light of this objective, the proposed method of seismic rating was selected because of its simplicity, flexibility and ability to yield reasonable results.

The vulnerability rating for a bridge must be performed with a minimum of computation, and with data readily available to the engineer. Two separate vulnerability ratings are currently proposed; the first for the bearings and the second for the remainder of the structure; namely columns, piers, footings, abutments and liquefaction potential. The greater of these two ratings will be the vulnerability rating for the structure. In areas of lower seismicity, only the vulnerabilty rating for the bearings needs to be considered.

Although, the engineer is allowed to used his judgement in performing vulnerability ratings, the guidelines do provide a suggested step-by-step procedure for arriving at each of the two proposed ratings. This procedure has its basis in the observations of the performance of bridge structures during past earthquakes. Structural configurations and details which have resulted in failure in the past are identified, and assigned vulnerability ratings between 1 and 10.

The seismicity rating will be based on the acceleration coefficient taken from the map shown in Figure 1.

The importance rating will be based on the social/survival and security/defense requirements used to establish the importance classification in ATC-6.[1] Essential bridges will be rated between 6 and 10 and all other bridges will receive a rating of 5 or below. Selection of the final importance rating from within this range of ratings will be based on the number of people likely to be on or under the bridge at any one time, and the relative importance of the bridge as a vital transportation link.

3.2 Detailed Seismic Evaluation

The detailed seismic evaluation of a bridge will be performed in two phases. The first phase will be a quantitative evaluation of individual bridge components using the results from one of the two analysis procedures developed for the ATC-6. The analysis will be performed using the design earthquake loading. The resulting force and displacement results, which are referred to as demands, will be compared with the ultimate capacities of each of the components to resist these forces and displacements. The ability of columns to resist post elastic deformations will be considered. A capacity/demand ratio will be calculated for each potential mode of failure in the critical components. This ratio is designed to represent the portion of the design earthquake that each of the components is capable of resisting.

The second phase of evaluation is an assessment of the consequences of failure in each of the components with insufficient capacity to resist the design earthquake. Consideration will be given to retrofitting substandard components if their failure results in a bridge collapse. In the case of certain essential bridges, the loss of function may also warrant the consideration of retrofitting. A procedure for selecting retrofit methods is shown in Figure 3.

There are four areas where local failure has a high potential of occurring and where component capacity/demand ratios will be calculated. These are:

- Bearings and Expansion Joints
- Columns, Piers and Footings
- Abutments
- Liquefaction of Foundation Soil

Aspects of the evaluation process relating to each of these areas are discussed in the following paragraphs.

3.2.1 Bearings and Expansion Joints

Bridge superstructures are often constructed discontinuously to accommodate anticipated superstructure movements such as those caused by temperature variation or to allow for the use of incompatible materials. Discontinuities necessitate the use of bearings which provide for rotational and/or translational movement. During earthquakes certain types of bridge bearings have proved to be among the most vulnerable of all bridge components.

In major earthquakes the loss of support at bearings has been responsible for several bridge failures. Although many of these failures resulted from permanent ground displacements, several were caused by vibration effects alone. The San Fernando, California earthquake of 1971[3], the Guatamala earthquake of 1976[4], and the Eureka, California earthquake of 1980[5] are some recent examples of earthquakes in which bridge collapse resulted from bearing failure. Even relatively minor earthquakes have caused failure of anchor bolts, keeper bar bolts or welds, and nonductile concrete shear keys. In many of these cases the collapse of the superstructure would have been imminent if the ground motion were slightly more intense or longer in duration.

Capacity/Demand ratios for bearings will be calculated for both displacement and force. Displacements are investigated in the longitudinal direction at expansion joints or "fixed" bearings where the force capacity is inadequate. The force capacity/demand ratio is calculated for bearings designed to resist lateral loads.

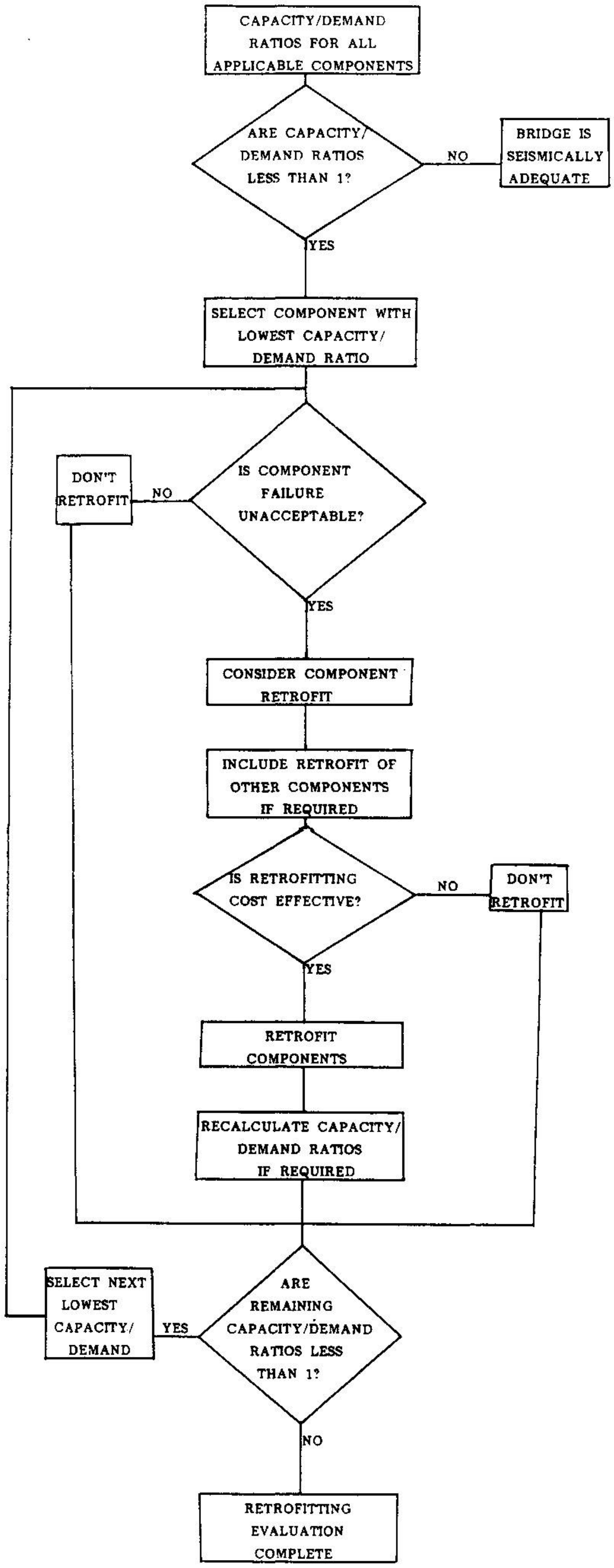

Fig. 3 Procedure for selecting retrofit methods

In the case of the differential displacements at expansion joints, elastic response spectrum analysis results yield displacements that are often below those intuitively expected based on observed bridge behavior during past earthquakes. In addition to the nonlinear behavior of expansion joints, possible independent movement of different parts of the substructure and out-of-phase movement of abutments and columns resulting from travelling surface wave motions also tend to result in larger displacements. For this reason, two methods for calculating the displacement capacity/demand ratio are proposed. The first method is based on a comparison of the nominal support length at the bearing (capacity) and the required design support length (demand) from the ATC-6. The second method compares the effective seat width (capacity) with the differential displacements obtained from an analysis (demand). Except in the case of restrained expansion joints where only the second method can be used, the lesser of the two capacity/demand ratios obtained from these two methods will govern.

The displacement capacity/demand ratio is intended to reflect the reduced level of loading at which a loss of support type of failure will occur. Usually the consequences of a support failure is the collapse of the span. In certain bridges with continuous superstructures, however, the bridge may still be capable of resisting the dead load moments and shears resulting from a loss of support at the expansion joint. This is often the case in long reinforced concrete slab bridges. Although a structure which has failed in this manner is not capable of carrying traffic loadings, it is likely that following a major earthquake it will be inspected and the expansion joint failure discovered. Traffic can then be diverted or measures taken to shore up the unseated bearings.

Conversely, certain structural configurations are exceptionally vulnerable to collapse in the event of a loss of support at the bearings. Such structures would be prime candidates for retrofitting. Simple or suspended spans in which no redundancy exists are particularly vulnerable. This is also true to a lesser degree in the case of a structure with a small redundancy, such as continuous bridges in which only one support occurs between expansion joints.

Elastic bearing forces obtained from a conventional analysis are likely to be lower than those actually experienced by bearings during an earthquake. This is because bearings, which are nonductile components, often do not resist loads simultaneously. This has been demonstrated in past earthquakes by the failure of anchor bolts or keeper bars on some, but not all of the bearings at a support. In addition, the yielding of ductile members such as columns can transfer load to the bearings. This phenomenon was observed in the results from nonlinear analytical case studies of three bridge structures(6). For these reasons it is necessary to increase elastic analysis force results through the use of a response modification factor when evaluating the force demand on nonductile motion-restraining components.

The force capacity of bearings must be carefully calculated. Anchor bolts are often subjected to combined bending and shear or high stresses at the threads. Spalling of edge concrete at anchor bolts is also possible. In addition, bearings may not be what they are represented to be on "As Built" plans or maintenance records.

By itself, the failure of bearing anchor bolts, keeper bars or shear keys will not constitute a situation that warrants retrofitting. When such a failure can result in relative displacements sufficient to cause collapse, however, then retrofitting should be considered. This determination should be made at the time that the consequences of component failure are assessed. For example, the loss of support of an edge girder due to transverse movement may render a portion of the superstructure unuseable but will not result in a structure collapse except possibly in a two girder bridge. It would still be possible to utilize the remaining portion of the superstructure. In this case retrofitting would not be warranted based on bearing force failure alone.

3.2.2 Columns, Piers and Footings

During an earthquake, the interaction of the columns and piers with their footings will determine the probable mode of failure for these components. The first step in evaluating these components is to determine if and where plastic hinging will occur. Plastic hinges may occur in the column end regions or at the footing. Piers can develop plastic hinges in the end regions about the weak axis only. The location of plastic hinging will dictate the modes of failure that should be investigated.

It is not uncommon for bridge columns to yield during strong seismic shaking. This is expected and provided for in the design of new structures. Existing columns however may not be capable of withstanding as much yielding as newly designed columns. Column failure may also occur prior to yielding in columns designed by pre-1971 standards. Column failures that result in a sudden loss of flexural or shear strength have the potential for causing collapse. The force levels at which these local failures occur will be reflected in the capacity/demand ratios for the various column failure modes. Each of these failures must be assessed in terms of its effect on the global stability of the structure. The cummulative effect of column failures elsewhere in the structure should also be considered in making this assessment.

Four modes of column failure are considered in evaluating columns. These include:

- Shear failure in the column
- Anchorage failure in the main longitudinal reinforcement
- Flexural failure in the column due to inadequate transverse confinement
- Failure of the splices in main longitudinal reinforcement

Detailed procedures are included in the guidelines for calculating the capacity/demand ratios for each of these modes of failure.

Column shear failures occur suddenly and can result in the rapid disintegration of the column. This happened to several bridges during the San Fernando earthquake. Flexural yielding of the column has the effect of limiting the shear force, but it also results in a degradation of shear capacity. The guidelines provide a technique for determining the level of yielding at which the danger of a shear failure is large. The level of yielding is represented by a ductility indicator which is applied to the flexural capacity. The capacity/demand ratio for column shear is then determined by comparing the modified flexural capacity with the elastic flexural demand.

In order to visualize this method, it is useful to look at the schematic relationship between shear capacity and shear demand as shown in Figure 4 for various levels of yielding. In this graph the level of yielding is indicated by the Ductility Indictor (D Factor). The shear capacity is shown as being constant up to a D factor of 2 where concrete spalling is assumed to begin. Between a D factor of 2 and a D factor of 5, the shear capacity is assumed to decrease linearly until it has reached the capacity of the effective transverse steel and the concrete core.

The applied shear force (demand) is proportional to the elastic moment up to flexural yielding (D=1). Beyond initial flexural yielding the shear force is assumed constant. To account for column overstrength, the maximum shear force is increased by a factor of 1.3. The point at which the shear capacity is equal to the shear force is the point representing the degree of flexural yielding at which shear failure may occur. If this occurs between a D value of 1 and a D value of 5, a reduced ductility capacity due to shear degradation in the column is indicated. This reduced ductility capacity is represented by the D value at which the shear capacity and shear force are equal in Figure 4. Therefore by evaluating the seismic capacity/demand ratio for flexure at this D factor the ratio of acceleration causing shear failure to design acceleration can be found.

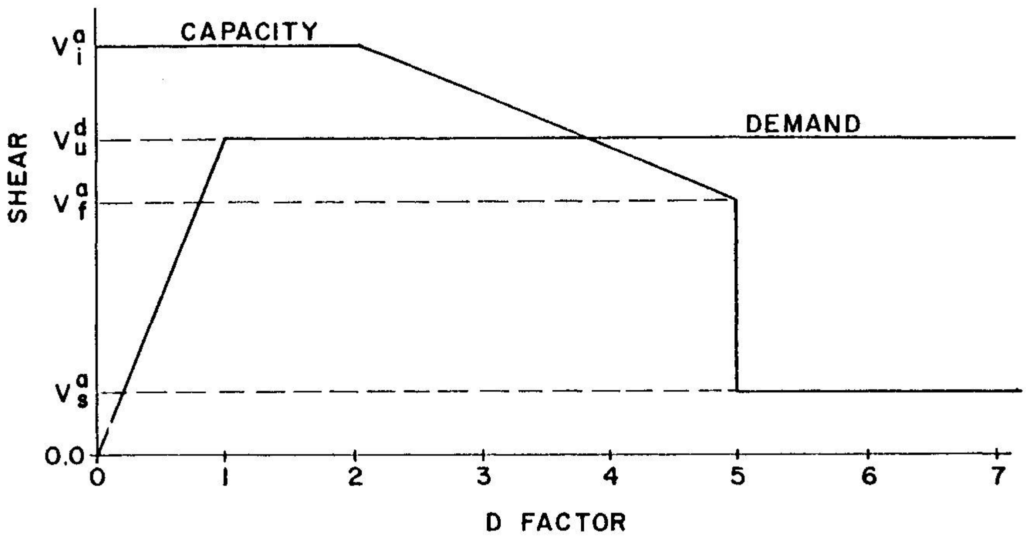

Fig. 4 Shear capacity and demand for reinforced concrete columns subjected to flexural yielding

If the initial shear capacity is less than the ultimate shear force, then the seismic capacity/demand ratio will be calculated as the ratio of the inital shear capacity to the elastic shear force caused by the design earthquake. If the final shear capacity is greater than the ultimate shear force resulting from plastic hinging of the column, then shear will not be considered a critical mode of failure. When yielding occurs in the footing, column shear capacity will not deteriorate, and shear failure may occur only if the ultimate shear force exceeds the initial shear capacity.

A sudden loss of flexural strength can result from an anchorage failure of the main reinforcement. This type of failure occurred at the Route 210/5 Separation and Overhead during the 1971 San Fernando Earthquake(3). When cracking occurs in the concrete where reinforcing steel is anchored, bond capacity is lost, and this type of failure is more likely. The procedures for calculating capacity/demand ratios for longitudinal steel anchorage will take this into consideration.

Sufficient transverse confining reinforcement is necessary to prevent strength degradation in flexure. In most existing columns the transverse reinforcement is not capable of preventing flexural degradation at the levels of yielding assumed in the design of new columns. Therefore, a method for determining the reduced levels of yielding at which existing columns will fail is proposed in the retrofit guidelines. This is also done through the determination of a ductility indicator that is applied to the ultimate flexural capacity of the column. This modified flexural capacity is divided by the elastic moment in the column to obtain the capacity/demand ratio.

The practice of splicing reinforcing bars at the bottom of the column was common in the past and may result in a high potential for failure during an earthquake. Flexural yielding of the column is likely to occur at this location, which will greatly reduce the capacity of the splices. The guidelines will consider this type of failure by limiting the amount of allowable yielding that can take place at a location where splices occur. Both anchorage and splice failures have the potential for limiting forces in the column. This may be critical in preventing column shear failures.

The capacity/demand ratio for the footing in flexure is calculated when yielding occurs in the footing. The allowable amount of flexural yielding will depend on the mode of footing failure. This is also represented by a ductility indicator that is applied to the ultimate footing flexural capacity.

3.2.3 Abutments

Failure of abutments during earthquakes usually involves tilting or shifting of the abutment either due to seismic earth pressures or inertia forces transmitted from the bridge superstructure. Usually these types of failures alone do not result in collapse or impairment of the structures capacity to carry emergency traffic loadings. They may result in loss of access, however, and can be critical in certain important structures.

Large horizontal movement at the abutments can result in approach fill settlements beyond acceptable limits. Abutment capacity/demand ratios therefore, are based on the abutment displacement. The displacement demand is assumed to be the elastic displacements at the abutments obtained by properly modeling the abutment stiffness. The displacement capacity is assumed to be three inches unless determined otherwise by a more detailed evaluation.

3.2.4 Liquefaction of Foundation Soil

Most foundation failures during earthquakes are the result of excessive soil movement such as occurs due to liquefaction. A capacity/demand ratio for liquefaction should be calculated when there is the potential for a severe liquefaction failure. This is obtained by dividing the ground acceleration at which liquefaction failure will occur by the design acceleration coefficient.

4. RETROFIT MEASURES

The guidelines will propose several conceptual details for retrofitting typical components that are known to be seismically deficient based on their past performance during earthquakes. These details are designed to prevent collapse or disabling structural damage due to the following modes of failure:

- Loss of support at the bearings which will result in a partial or total collapse of the bridge
- Excessive strength degradation of the supporting components
- Abutment and foundation failures resulting in loss of accessibility to the bridge

Once strengthening of a component has been decided upon, the guidelines will recommend that component retrofitting be designed to the standards for new construction. Reduced levels of seismic retrofitting may be considered when it is not cost effective to retrofit to new design standards and partial strengthening will greatly reduce the chances for structure collapse.

4.1 Bearing and Expansion Joint Retrofitting

Several techniques for retrofitting bearings are proposed. These include:

- Longitudinal Joint Restrainers
- Transverse Bearing Restrainers
- Vertical Motion Restrainers
- Bearing Seat Extensions
- Replacement of Bearings
- Special Earthquake Resistant Bearings and Devices

Longitudinal Joint Restrainers are used extensively by the California Department of Transportation(7). The primary function of these devices is to limit relative displacements at joints and thus decrease the chances for a loss of support at these locations. Restrainers are designed to resist force in the elastic range. Careful attention must be given to the methods used to attach restrainers to the superstructure so that existing components will not be damaged during an earthquake. The typical retrofit detail used by the California Department of Transportation on its concrete box girder bridges is shown in Figure 5.

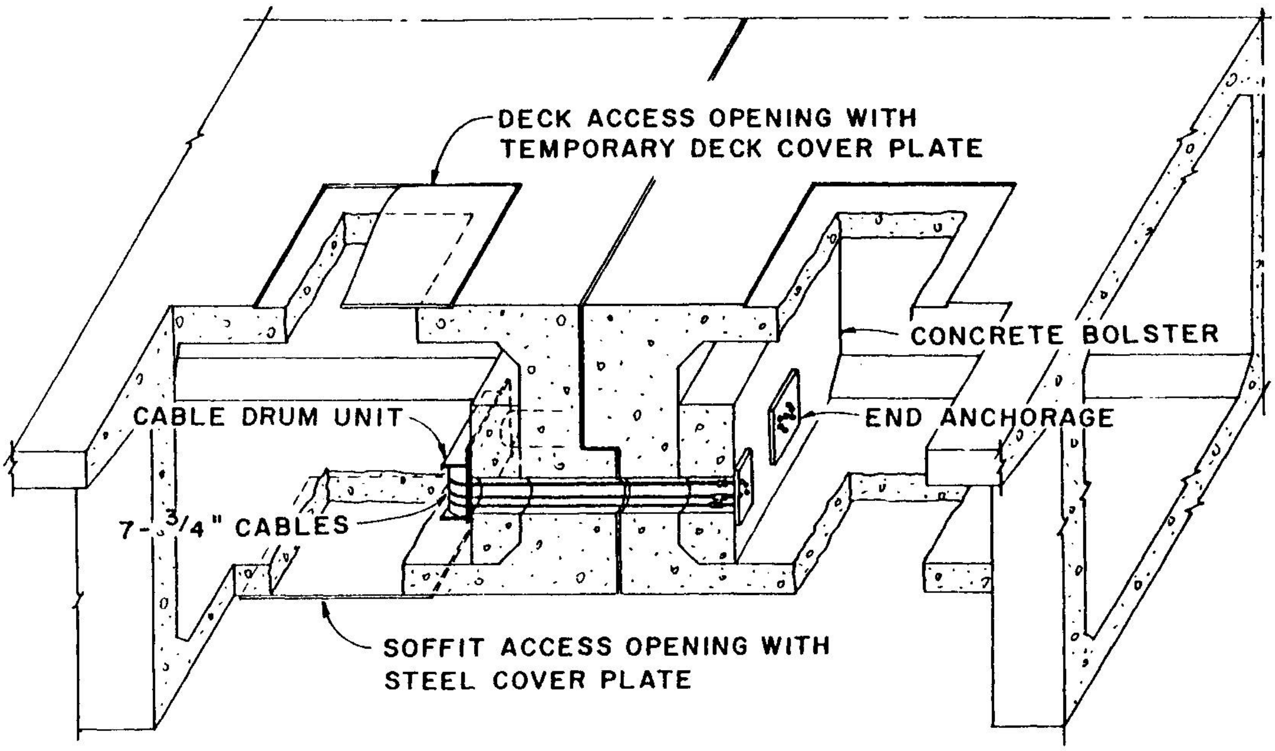

Fig. 5 Longitudinal joint restrainer for concrete box girder

Two other types of restrainers utilized at bearings and expansion joints are designed to restrict either transverse or vertical motion. Transverse bearing restrainers should be used when the existing anchor bolts, shear keys, or keeperbars are inadequate to resist transverse forces and when a loss of support due to transverse motion is likely due to the structure configuration or bearing support details.

The need for vertical motion restrainers will seldom be demonstrated by an analysis. However experience has shown that vertical movement can take place at the bearings. This can lead to the displacement of bearings and possibly increase the chances of a loss of support failure. The guidelines recommend that vertical restrainers be installed if feasible whenever longitudinal restrainers are considered as a retrofit measure and the seismic uplift force obtained from an analysis of longitudinal motion exceeds fifty percent of the deadload reaction.

Bearing seat extensions may be a feasible retrofit measure in certain situations. Extensions allow larger relative displacements to occur at the joints before support is lost and the span collapses. Since high forces may be imposed on these extensions, it is recommended that if feasible, such as at abutments, they be supported directly on the foundation. When this cannot be done, such as at columns or piers, bearing seat extensions should be designed using substantial overload factors.

Bearings which are damaged or malfunctioning can fail during an earthquake. In addition certain types of bearings, such as those shown in Figure 6, have performed poorly during past earthquakes. A possible retrofit measure in these cases is the replacement of the bearings with modern types such as elastromeric pads which, in conjunction with adequately designed restrainers, are more effective in resisting seismic loading.

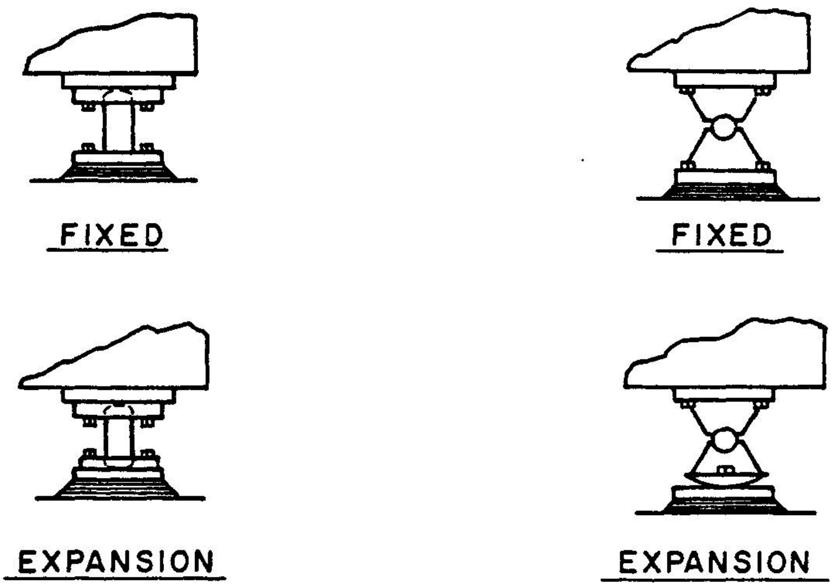

Fig. 6 Seismically vulnerable bridge bearings

Certain types of bearings and devices have special performance characteristics which will alter the seismic response of the entire structure. Some of these are designed to act as force limiting devices which minimize the force that can be transferred to supporting columns, piers or abutments. The behavior of these devices is usually highly non-linear. Some of the devices have been extensively tested but none of those currently installed in new bridges have been subjected to actual earthquakes. The guidelines recognize the complexity of these devices and recommend that a special design be performed if they are to be used as a retrofit measure.

4.2 Column, Pier and Footing Retrofitting

Columns, piers and footings may fail in any of several ways during an earthquake. In general it is more difficult and less cost effective to specifically retrofit these components than it is bearings. However, if force-limiting bearing devices can be added between the superstructure and columns, piers, or abutments, a cost-effective retrofit measure can be achieved without the necessity for retrofitting the substructure. This method of retrofit has been proposed for use in New Zealand(8).

To date there are very few retrofit methods that have actually been tried on seismically deficient bridge columns. Several methods have been proposed, however, and these are discussed in the following paragraphs.

Improved confinement will increase the ability of a column to withstand repeated cycles of loading beyond the elastic limit and tend to prevent column failure due to shear, pullout of longitudinal reinforcement, and degradation of flexural capacity. The ATC "Seismic Design Guidelines for Highway Bridges" have requirements for the spacing, amount and anchorage of conventional transverse reinforcement. The use of conventional transverse reinforcement for retrofitting, however, would present construction difficulties and would be of questionable effectiveness. Several methods of increasing the transverse confinement of columns through retrofitting have been proposed.

The first method utilizes conventional half inch steel reinforcing that is prestressed on the outer face of the column through the use of a specially designed turnbuckle. The steel bars would be spaced at 3½ inches on center which would provide confinement equivalent to new construction in most cases. The steel would be protected with a layer of pneumatically applied concrete.

A second method is similar to the first except that quarter inch prestressing wire is wrapped under tension around the column. A method of anchoring the wire would be required. The wire and anchorages would be protected by the same technique used for the first method.

The third method would employ a solid steel shell that would be welded in place around an existing column. A small space would be left between the column and the shell that would be grouted solid. The steel could be of a weathering type or it could be ordinary painted steel.

Other methods of increasing the confinement of concrete members have been tested in the laboratory at the Georgia Institute of Technology(9). In one of these methods, steel banding of the type used for packaging materials was applied to the outside of the concrete member. This method made a definite improvement in the ability of the concrete member to withstand repeated cycles of yielding. Because of the limited sizes of available banding, it is questionable if this method would be effective for the larger sizes of bridge columns, but tests have proved its effectiveness for the smaller sizes.

The Japanese have also proposed several methods for increasing the transverse confinement of reinforced concrete building columns which could be used for many smaller bridge columns. It should be stressed that retrofitting to increase transverse confinement has not been tried on an actual structure and with the exception of methods tested at the Georgia Institute of Technology and the Japanese methods, no physical tests have been performed. The advantages of confinement are well established by physical testing, however, and any method that can increase the confinement should be considered as a potential retrofit measure.

The maximum shear force on a column can be reduced by decreasing the yield moment at one or both ends of the column. This can be done by cutting the longitudindal reinforcing bars. Since this will increase the ductility demand at the points of flexural yielding, this retrofitting technique must be employed with caution. An essential prerequisite to using this retrofit method is that loss of flexural capacity at the location of cut bars not result in an overall structure instability, since any uncut bars at this location can be expected to yield in the early stages of seismic shaking.

This retrofit method should be considered when columns are overreinforced for flexure resulting in little or no flexural yielding during an earthquake. The resulting high yield moments could produce shear forces above the capacity of the column. By cutting bars an increased amount of yielding is accepted in exchange for a reduced shear force. The net result could be an improvement in the overall earthquake resistance of the structure.

The use of increased flexural reinforcement has also been proposed(10). The retrofit technique will increase the flexural capacity of the column. Increased flexural capacity will increase the forces transferred to the foundation and the superstructure/column connections and will also result in an increased column shear force. In addition the strengthened column will be stiffer and thus may attract more seismic force. If increased flexural reinforcement is being considered, care should be taken that all other components are able to resist the forces developed by the strengthened column. Since failure of the footings or failure of the columns in shear is usually more critical than excessive flexural yielding, this retrofit technique should be used with care. This technique should only be considered when loss of flexural strength would result in a collapse mechanism being formed and when levels of yielding in the column are exceptionally high.

In many cases the column footing will fail before the column or pier yields. This is often due to the absence of a top layer of footing reinforcement capable of resisting uplift forces on the footing. During an earthquake this can result in the fracturing of the concrete and the loss of anchorage for the longitudinal bars. This condition is usually most critical in single column bents.

One suggested method of retrofitting columns with this type of deficiency involves a concrete cap of constant thickness and the same horizontal dimensions as the footing which would be cast directly on top of the footing. Continuity with the existing footing would be provided by steel dowels cast in drilled holes. Negative moment capacity would be provided by a top layer of conventional reinforcement and prestress tendons. The collar would strengthen the footing for uplift and provide an extra measure of confinement at the base of the column and the top of the footing.

4.3 Abutment Retrofitting

Abutment retrofitting tehniques are suggested which tend to prevent loss of access to the bridge. These techniques are usually justified for structures which serve a critical function. Abutment tie back systems and settlement slabs are the only two abutment retrofit measures discussed in the guidelines.

4.4 Retrofitting for Liquefaction

Liquefaction or excessive soil movement has been the cause of many bridge failures during past earthquakes[11,12]. There are two suggested approaches to retrofitting that will mitigate these types of failure. The first approach is to eliminate or improve the soil conditions that tend to be responsible for seismic liquefaction. The second approach is to increase the ability of the structure to withstand large relative displacements similar to those caused by liquefaction or large soil movement. The first approach has been tried on dams, power plants and other structures but to date has not been used as a retrofit measure for bridges. The second approach will utilize many of the retrofitting techniques discussed previously.

Several methods are available for stabilizing the soil at the site of the structure. Each method should be individually designed using established principles of soil mechanics to insure that the design is effective and that construction procedures will not damage the existing bridge. Some possible methods for site stabilization include:

1. Lowering of Groundwater Table
2. Consolidation of Soil by Vibrofloatation or Sand Compaction
3. Vertical Network of Drains
4. Placement of Permeable Overburden
5. Soil Grouting or chemical injection

Some of these methods may not be suitable and may even be detrimental in certain cases. Therefore, careful planning and design is necessary before employing any of the above site stabilization methods.

Any method that will tend to prevent loss of support at the bearings will be useful in preventing structure collapse due to excessive soil movement. Therefore most of the methods for retrofitting bearings should be considered in a structure subjected to excessive soil movement. In addition, the ability of the substructure to absorb differential movements is important. If, for example, column shear is the critical failure mode, retrofitting methods such as cutting longitudinal reinforcing steel that will tend to make flexure the dominant failure mode should be considered. Usually retrofitting of the structures alone will not prevent severe damage. Retrofitting is intended to prevent collapse and possibly provide for some restricted use of the structure immediately following an earthquake.

At a site subjected to excessive liquefaction, methods to improve the structure may be ineffective unless coupled with methods to stabilize the site.

5. CONCLUSIONS

The seismic retrofitting of bridges is still very much an art. Because of the variety of bridge configurations and details it is difficult to specifically cover all cases by a guideline. The guidelines are being developed to insure that engineers contemplating bridge seismic retrofitting have the benefit of the experience gained to date. In addition, the guidelines are being written to encourage innovative thinking and to aid in advancing the state of the art. They are written in a rational framework so that new ideas can be easily incorporated in future updates.

The development of the guidelines is still in progress and it is likely that modifications and additions will still be made. When completed, it is hoped that the guidelines will provide engineers with the information needed to solve the problem posed by our seismically deficient bridges.

REFERENCES

1. APPLIED TECHNOLOGY COUNCIL, "Seismic Design Guidelines for Highway Bridges", October 1981.

2. CALIFORNIA DEPARTMENT OF TRANSPORTATION, "Bridge Design Specifications", October, 1981.

3. FUNG, GEORGE G.; LeBEAU, RICHARD J.; KLEIN, ELDON D.; BELVEDERE, JOHN; AND GOLDSCHMIDT, ADLAI F., "Field Investigation of Bridge Damage in the San Fernando Earthquake", State of California, Division of Highways, Bridge Department, 1971.

4. COOPER, JAMES D., "Bridge and Highway Damage Resulting from the 1976 Guatemala Earthquake", FHWA, Washington, D.C., Report No. FHWA-RD-76-148, May 1976.

5. SEMANS, FRANK AND ZELINSKI, RAY, "Final Report of the Eureka (Trinidad - Offshore) Earthquake of November 8, 1980", Caltrans, Office of Structures Design.

6. IMBSEN, R.A.; NUTT, R.V. AND PENZIEN, J., "Seismic Response of Bridge Case Studies", Report No. UBC/EERC - 78/14, Earthquake Engineering Research Center, University of California, Berkeley, June 1978.

7. DEGENKOLB, ORIS H., "Retrofitting of Existing Highway Subject to Seismic Loading-Practical Considerations", Proceedings of a Workshop on Earthquake Resistance of Highway Bridges, Applied Technology Council, Report No. ATC-6-1, November 1979, pp. 343-359.

8. BLAKELEY, R.W.G, "Analysis and Design of Bridge Incorporating Mechanical Energy Dissipating Devices for Earthquake Resistance", Proceedings of a Workshop on Earthquake Resistance of Highway Bridges, Applied Technology Council, Report No. ATC-6-1, November 1979, pp. 313-342.

9. KAHN, LAWRENCE F.; SRIANO, B.J., "Improving Ductility of Existing Reinforced Concrete Columns", Proceedings of the 2nd U.S. National Conference on Earthquake Engineering, Earthquake Engineering Research Institute, Aug., 1979.

10. ROBINSON, R.R; LONGINOW, A. AND ALBERT, D.S., "Seismic Retrofit Measures for Highway Bridges", Vol. 2: Design Manual, FHWA, Washington, DC, Report No. FHWA-TS-79-217, April 1979.

11. FERRITTO, J.M. AND FORREST, JB., "Determination of Seismically Induced Soil Liquefaction Potential of Proposed Bridge Sites" - Volumn I & II, Federal Highway Administration, Department of Transportation, August 1977.

12. MARTIN, GEOFFREY R., "Seismic Design Considerations for Bridge Foundations and Site Liquefaction Potential", Proceedings of a Workshop on Earthquake Resistance of Highway Bridges, Applied Technology Council, Report No. ATC-6-1, November 1979, pp. 206-227.