

FOURTH WORLD CONGRESS ON JOINT SEALANTS AND BEARING SYSTEMS FOR CONCRETE STRUCTURES



EDITOR
BARRIE ATKINSON



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The Fourth World Congress on Joint Sealing and Bearing Systems for Concrete Structures

Editor

Barrie Atkinson



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PREFACE

Although bearing and joint systems comprise only a small percentage of the construction cost of highways, buildings, and bridges (1 percent, typically), their importance to the functioning of those structures is vastly greater. For without adequate engineering effort in design of these vital elements to account for the many, sometimes conflicting, requirements of loading and/or movement, the integrity of the entire structure may be compromised.

Our increased understanding of seismic activity has made us more aware of the wider area of this activity as well as the greater demands placed on support and joint systems.

This Fourth (quinquennial) World Congress on Joint Sealing and Bearing Systems in Concrete Structures will enhance the general understanding of these systems and introduce entirely new concepts developed to cater to the latest seismic code requirements.

As with the first two Congresses, this is V. 1 of a planned two-volume set. V. 1 contains 22 papers, plus the abstracts of papers presented at the Third Congress. V. 2 will be published following the Fourth Congress.

Barrie Atkinson
Chairman, IJBRC

**The Fourth World Congress on Joint Sealing
and Bearing Systems for Concrete Structures**

**Sacramento, California
September 29-October 3, 1996**

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A Half-Century of Involvement with Joints and Bearings and Some Lessons Learned

by Stewart C. Watson

From shortly before the entry of USA in World War II and to the present, I have been continuously involved in the design, testing, manufacturing and observation of the performance of joints of all types from pavements to bridges and bearings of all types from the old rockers to elastomeric, pot, disc and then to earthquake isolation concepts. Starting out with load transfer devices buried in concrete pavement joints for state highways and airfield pavements to field molded sealants and then compression seals, the design trend in pavements has been from longer 100 feet panels (30m) to relatively short panels of 15' (4.5m). This has greatly simplified the sealing problem since the distance changes between joint interfaces of shorter length panels obviously are much less in creep-shrink and thermal volume change.

With respect to bridges, the design trend has been reversed going from relatively short decks (40' (12m)) to longer and longer spans greatly complicating the sealing problem. It was in this confused design period that the writer worked towards developing sealing and bearing systems for every conceivable type pavement or bridge structure. Some lessons learned during the past 50 or more years, are the subject of this paper.

Keywords: Bridge bearings; bridges (structures); joints (junctions); pavements; sealing

INTRODUCTION

Sealing of Pavement Joints from 1945 - 1960

Most of the early attempts to seal 100' (30m) long pavement joint panels with field molded sealants of that era were a dismal failure. Variations of rubber and asphalt materials proved inadequate to handle the movements of the 78 foot long (23.8m) State of New Jersey pavement panels or the New York or Illinois 100 foot (30m) concrete slab length design. Other states incorporated slightly shorter length slabs in their pavement design, but the movement was just too much for the existing sealant compounds of that era to work effectively. During the late 1950's, the writer was asked to accompany an inspection team surveying the performance of various types of load transfer devices from one end to the other of the entire New York Thruway. It was astonishing to observe that not one single cement concrete pavement joint on the entire New York Thruway (100 foot panels - 30m) which incorporated field molded sealants of the rubber asphalt and polysulfide types was performing as intended in spite of the claims of manufacturers for 200% - 400% elongation as defined in ASTM and Federal specifications. It was this eye opening experience in 1955 which laid the groundwork for the introduction of the preformed neoprene compression seal.

Experiments with the Compression Principal - 1958 - 1960

The writer and Thomas Bowman, after considerable experimentation with various formed in place sealants, attempted to include the compression principal into liquid applied sealants by developing a material that when poured into a typical joint, would quickly skin over and then begin a period of foaming which tended to place that material initially at least into compression. While the short term result was impressive, it was difficult to control the foaming action so that the sealant would mound up over the joint surface of the pavement. Later it developed that the resultant sealant plug was lacking in the necessary durometer and elongation properties for long term performance.

With the compression principal still in mind, we then began to experiment with various extruded rubber shapes such as round tubes, box shaped and chevron shaped tubes which finally evolved into a rectangular shape that included a cross braced internal web design, a recessed folding top and a pointed bottom that was necessary for insertion into the joint. In an attempt to use the finest rubber in the state of the art at the time, we decided upon Neoprene because of its proven outdoor performance properties most of which were in successful use in electrical power line construction.

The Fuller Road Test - 1960

In an attempt to confirm our laboratory and design thinking, five 12' (3.6m) long joints were installed in a cement concrete pavement of Fuller Road in Albany, New York by the writer and T.C. Bowman in the Fall of 1960 with these new experimental extruded compartmented neoprene extrusions. As a control, polysulfide sealants were used on the adjacent pavement lane so that a clear comparison of performance could be made. The neoprene seals were continued up through the right angle curb so that the performance at these directional changes could be observed as well.

Condition surveys, in the Spring of 1961, indicated a clear performance superiority of the neoprene seals over the polysulfides. The State of New York, faced with wholesale failures of field molded sealants all over the state, issued a directive that the neoprene compression seals could be substituted as an alternate to field molded sealants if the comparative costs were in their favor. The result was that contractors switched over to neoprene compression seals on an extensive basis.

Early Experience with Neoprene Seals - 1961 - 1962

In their first year of wholesale usage, a number of failures occurred primarily from the lack of quality in the neoprene extrusions since there existed no field proven specification. A number of rubber firms who had quickly addressed themselves to its manufacture neither cared nor had any conception of what constituted quality in a neoprene extrusion. To make things worse, no one at that time really knew what constituted quality in a neoprene compression seal.

In one case where 12 feet (3.6m) wide pavements were being constructed, the first lane in the summer and the second in late fall, it became apparent that the compression seals placed in the summer placed pavement became loose in the winter where as the seals placed in the cooler time of the year stayed in compression nicely during the winter. They were placed in 60' (18.3m) long pavement panels and the original joint width was 3/8" (10mm). 11/16" (17mm) wide seals were installed in these joints and it was observed that the pavements constructed in the hot summer exhibited a total joint width change of 3/8" to 1/2" (10mm to 13mm) which was a combination of high temperatures, shrink and thermal movement. The seals that came loose were replaced with wider 13/16" (20mm) seals and they worked well through that next winter season.

The lesson learned here has long been remembered by the writer particularly as these seals began to see use on concrete bridge decks later on, is that in the sealing of joints particularly on concrete structures, one must always be cognizant of the as placed temperature width of the joint opening.

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The Del Mar By Pass Test - 1963

As the extruded neoprene seals began to completely take over the sealant needs of New York State, considerable static from the field molded sealant fraternity had such a sobering effect on research personnel of the New York State Department of Public Works, that to settle the argument once and for all, they began to organize one of the most comprehensive full scale tests of sealing systems in the history of highway construction.

In June of 1963, some 37 different types of pavement seals representing the products of 21 manufacturers were installed on the Del Mar Bypass near Albany, New York under the supervision of the New York State DOT Bureau of Physical Research. Only manufacturers were permitted to install their sealants and each sealing product was allotted 5 transverse contraction joints to seal. Every conceivable type of field molded polysulfide, polyurethane, cold poured rubber asphalt, hot poured rubber asphalt, latex with extender, adduct rubbers, liquid neoprene and tar based sealant together with preformed silicone, EPDM,

polyurethane foam and preformed compartmented neoprene seals in the state of the art was installed.

In 1961, 1963, 1964 and 1965 a series of interim reports were published by the New York State DOT Bureau of Physical Research all highly favorable to preformed compartmented neoprene seals. Then came a final report (1) in December of 1968 which concluded that after ten years of research and field experience with sealers "their investigation has revealed that preformed neoprene is the only sealer that preformed satisfactory for more than 3 years". The report indicated that while they had experienced some compression set in some of the seals, that it was attributed to the fact that too small of a seal was used and their pavement design for contraction joints was then modified to require a 5/8" (16mm) wide as constructed contraction joint and a 1-1/4 (32mm) wide compression seal. The Del Mar Bypass test was followed by tests in numerous other states and authorities such as Pennsylvania, Ohio, Michigan, Minnesota, North Dakota, Florida, Nebraska, Colorado and Kentucky with similar conclusions.

Bridge Compression Seals

In 1962, we had begun to bond two pavement sized compression seals side by side to obtain the greater width needed for bridge expansion joints and subsequently commenced to extrude them as one single extrusion. It then became known that bridge engineers who were also experiencing very high failure rates with field molded sealants began to see the wisdom of switching over to these large compression seals. The first bridge size compression seal

was installed on a bridge near Albany, New York and the second on a bridge in New Jersey. Michigan followed and subsequently over a period of time, virtually every state in the union began their use as well as many Canadian Provinces. Of course different manufacturers began to enter in to and contend for this vast new market and inevitably early failures began to surface for a great variety of reasons but primarily due to premature compression set.

Ultimately a good preformed compression seal specification was arrived at primarily due to the efforts of the State of Minnesota DOT Research Department. The number of manufacturers levelled off with only a few capable of meeting this new rather difficult specification and the field performance significantly improved in terms of compression set resistance.

The lesson learned here was that all organic materials, when subjected to prolonged tension or compression, will be affected by a change in shape or compression set. The degree of deformation or compression set is a function of the severity and elongation of the tension or compression force which that device is subjected to. The originators of neoprene claimed that like a bearing pad in compression which has a demonstrated long life in service, that a neoprene seal would last just as long. What they did not know or understand at that time was that when a cross braced compartmented compression seal is squeezed in compression, its internal webs which supply the pressure are actually bent in tension and this was the root cause of many of the early field failures. Obviously, the quality of the rubber compound in the extrusion also played a part but the principle of a compression seal whose internal webs are bent in a tensile mode for long periods of time must face up to a relatively short service life.

Bayshore (2) of the State of Michigan Research Department then reported a measured loss in pressure for compression bridge seals of as much at 70% - 80% in just 2 years which for all time changed the predicted life of a compression seal to something less than what its originators hoped it would be. Some researchers even today lay claim to its effectiveness under ideal conditions as ten years. Strangely enough, the remaining seals in the original 1960 Fuller Road were still performing effectively after a 30 year period of time.

The lesson learned here was that since all compression seals would incur compression set regardless of the quality of the vulcanizate, that it would be best to use a very high type of adhesive to install them rather than a lubricant adhesive which was then in wide use. Bridge compression seals are today installed with excellent adhesives which have tended to extend their performance life to some degree. Cerrating of the compression seal walls to improve their bond strength has also been resorted to by some manufacturers.

We made attempts to switch pavement compression seals over to higher type adhesives but since some members of the industry chose to continue on with

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lubricant type, it still prevails in use today.

Mechanically Locking of Compression Seals

With the knowledge that bridge compression seals in particular needed every possible chance to perform as a leak proof joint, we began to introduce mechanically locked seals as shown in Figure 1. This was a significant improvement over plain bridge compression seals but a bit more costly to fabricate so they did not catch on well in North America because some suppliers were still arguing that their particular vulcanizate was of such high quality that there was no need for mechanical locking.

Strip Seals

In early 1970, with the full realization that the compression principle of sealing joints on bridges tended to be short lived, the writer introduced to North America what today is known as strip sealing. Adapted from German thinking, the product of the mind of Waldemar Koster, the strip seal represented a marked departure from bridge compression sealing then in wide use as well as more economical methods. Steel extrusions were introduced which at that time were only available from hot press firms in Germany. The process was to heat a steel billet to white hot temperatures, dip it in powdered glass for lubrication and then force it through glass dies into the desired shape. Figure 2 illustrates the type of extrusions that we introduced into the market in the early 1970's. They caught on like wildfire and today they still virtually control the North American market after 20 years in wide usage.

There are however some serious problems with strip seals and one lesson learned here has to do with the difficulty in maintaining tolerance controls in the steel extrusions as well as tolerance controls in the neoprene rubber extrusions. The resultant problem was either a difficulty in installing the extrusions in the steel claws or what was much worse, a tendency to looseness with the neoprene rubber extrusions working free in field service. High performance adhesives were then introduced but this does not entirely solve the problem. Another problem is the ripping or cracking of the neoprene rubber extrusions in field service which is probably due to the stiffening of the rubber at lowered temperature. Also the very deep fold in the rubber extrusions which with 3" - 4" (75mm - 100mm) movement sealing glands comes close to 4" - 5" (100mm - 125mm) and tends to attract large quantities of incompressibles which cannot be ejected. Ice buildup over the top at lowered temperatures tends to damage the seals under high speed repetitive truck loading.

Molded Elastomeric Rubber Cushion Systems - 1968 - 1980

From the period of about 1968 to 1980, molded elastomeric rubber cushion

tension compression seals came into very wide usage. Originally introduced by the General Tire and Rubber Company, they became so overwhelmingly popular that many bridge engineers absolutely refused to consider the use of anything else.

The lesson learned here was that if you wanted to be in the bridge seal business in the 1970's, you had to come up with a rubber cushion of some type. Very strong patents had been obtained which made the entry of competition next to impossible without inviting a major patent law suit by a very large rubber firm.

The first and original rubber cushion device is illustrated in Figure 3. A second is shown in Figure 4 and these two systems dominated the bridge market for well over a decade however the introduction of a second type resulted in costly litigation which lasted for many years.

These devices worked fairly well structurally however very high stresses were at work during their tension and compression phase of movement. Bolts tended to work loose, these large moldings tended to delaminate from their imbedded metal plates and snow plows reeked havoc with them if they were even slightly exposed to traffic. The most serious problem was that they leaked badly at the juncture of each molding even though they were of tongue and groove design and installed under compression. The curb and gutter connections tended to leak very badly particularly on skewed joints and this problem, after it became fairly well known in the trade, tended to initiate their demise. However a great many of them were installed on North American bridges and some of them are still in service today.

The writer whose firm produced a rubber cushion joint attempted to encourage the rest of the industry involved to install a continuous sheet of rubber underneath the rubber cushions from curb to curb but to no avail. It remains a question still today with these rubber cushion devices which were so overwhelming popular for a decade or so, whether they would still be in use if the installation of a rubber sheet underneath would have solved their leaking problem.

Rubber Cushion Strip Seal Systems

The Felt Products Company in the late 1970's introduced a modified rubber cushion system which incorporated rubber end dams together with an integral rubber sealing gland that was bolted into place over a prepared blockout in the bridge deck. This system was installed in sections 4' (1.2m) long with end flaps at each end to end connection. Essentially the field problems were the same as the previously introduced rubber cushions such as snow plow damage, etc. Because of the relatively short lengths, they leaked profusely. The writer introduced two similar versions with molded lengths up to 10' (3m) to minimize

the number of joints but even so, they leaked at the end to end connections.

The lesson learned here was that molded sections installed with relatively short pieces in the field all tend to leak regardless of the type of connection. The primary function of a bridge expansion joint is to prevent leaking of deleterious chlorides on to the substructural elements below and so all of the rubber cushion systems have since gone completely out of use because they failed to accomplish this fundamental task.

Movement Related Failure Modes

As our structures and spans have become thinner, lighter in cross section (hollow boxes as compared to massive sections), and the economies from post-tensioning have been realized, the old historical movement data accumulated by local bridge design offices is often inadequate and outdated. These new more modern designs no longer enjoy the safety factor of thermal inertia or thermal lag and the thinner members tend to permanently shorten in creep-shrink in a magnitude that has surprised many experienced bridge designers. In fact, some of our newest, most technically exciting bridge structures have experienced total rupture of the expansion joints that have literally been torn apart in less than 2 years service.

It must be remembered that temperature gradients within the mass of newly placed bridge members vary widely by as much as 50 degrees F (28 degrees C) due to hydration which is a function of type and amount of cement, thickness of cross section, temperature of mixing water, surrounding air and from insulation. Other factors such as overall geometry of the bridge cross section, latitude and altitude, orientation of the bridge axis with respect to the sun, time of day and season, cloudiness or turbidity of the atmosphere, diurnal variations in ambient temperature, wind speed, nature and color of deck surfaces with respect to solar radiation absorptivity, emissivity and surface convection coefficient, thermal and physical properties of the constituent materials in the bridge, thermal conductivity, specific heat and density, ad infinitum.

It would be next to impossible with such a variety of variables for the average designer of bridges to make any accurate value judgement so it behooves those charged with the care of public funds to utilize healthy movement safety factors. It is the writers considered judgement that the following movement provisions are in the safe range for North American structures based on a 100 degrees F (56 degrees C) range of temperature environments:

1/8" (3mm) of movement for each 10' (3m) of deck length for thermal. Add 100% for creep-shrink and other phenomena.

Movements Other Than Thermal

Rubber cushions and most other types of sealing systems are designed to take a given amount of movement after which they will fail. A great many bridge authorities tend to disregard this or fail to provide for long term movements other than thermal and the expansion joint supplier tends to be blamed for failures which many times are not completely his fault.

Your attention is called to the interesting work of Moulton entitled "Observations of Highway Bridge Movements and Their Effects on Joints and Bearings" reported to the 1983 Session of the U.S. Transportation Research Board (3). In a comprehensive measurement study of movements other than thermally induced, on hundreds of bridges throughout 39 states and 4 Canadian provinces, vertical and horizontal abutment displacements averaging from 3.7 to 6.9" (94mm to 175mm) were recorded (Figure 5, Table 1) which is enough to damage or destroy any concept of bearing or expansion joint. In Figure 5, Table 2 we see average vertical and horizontal pier movements being in the area of 2.5 to 5.1" (63 to 129mm), and while these extra movements over thermal resulted in a multiplicity of other failure mode phenomena, 34 actual cases of damage occurring to the bearings were recorded (Figure 5, Table 3).

If Professor Moulton's summary is valid, the present movement rating practices of those 39 states and 4 Canadian provinces studied should result in future damage to bearings in the magnitude of 17% of their bridges. Figure 5 illustrates the magnitude and causes of these movements and the lesson learned here by the writer which came rather late in his career would have saved a small fortune in replacing and rebuilding bridge expansion joints in service which were way under designed for the actual movements that came. These movements which added to the thermal coefficient, tore apart many mechanical expansion joints. Another lesson to be learned here based on Professor Moulton's findings is that rather husky extra movement potential, which unfortunately costs a lot more money, should be added to the thermal prediction for all bridge expansion joint requirements.

Aluminum Armored Expansion Jointing Systems for Smaller Movements

From the period 1975 to 1985, the use of aluminum as the structural component for bridge expansion joints came into wide usage. High strength aluminum manufactured to grade ASTM 6061-T6 or its equivalent was the material of choice by most manufacturers. Figure 6a and Figure 6b denote some of the configurations that were used. Normally neoprene extrusions were used however in some of the colder climates, natural rubber came into use. Some type of locking claw similar to steel armored strip seals would be used to fasten the rubber component to the aluminum extrusion.

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While they looked good in the installation process, were light and easy to handle in the field, they somehow did not have the damping properties of steel and many of them came loose from their fastenings particularly if expansion bolts were used.

The crowning feature of aluminum was its apparent resistance to corrosion as compared to steel used in expansion joints. These aluminum extrusions were often treated with factory applied anodized coating which made them even more resistant to oxidizing effects.

One of the disadvantages of aluminum learned from use in the field was attributed to its coefficient of thermal expansion which is roughly three times that of concrete or steel. Figure 7 illustrates the problem which shows a rather substantial gap in the aluminum edge armor of an in service expansion joint. The two pieces of the aluminum extrusion were bonded in the concrete end to end to form the edge armor of the system. It is obvious however that the marked difference in the expansion coefficients of the concrete and aluminum resulted in this leaking crack at the very edge of the expansion jointing system. This crack was a working crack that allowed the penetration of deleterious chlorides into the deck ends and substructural components of the bridge which defeats the whole purpose of a sealed expansion joint.

The inherent disadvantages of at least some of these aluminum armored systems as compared to steel tended towards the declining use of aluminum for bridge expansion joints in USA so that today they are rarely used. The advantage of aluminum in corrosion resistance apparently could not overcome the much lower cost of galvanized or painted steel at least in the view of the various manufacturers who addressed themselves to the making of bridge expansion joints so that today, few if any aluminum expansion joints are being marketed.

Aluminum Armored Expansion Jointing Systems for Larger Movements

Larger movement aluminum expansion jointing systems came into being around the late 1970's and continued on for about a decade or so. The same advantages and disadvantages of aluminum armoring still prevailed as in the case of smaller movement aluminum armored systems. Figures 8 and 9 illustrate two popular aluminum modular systems.

The system in Figure 8 incorporated neoprene rubber extrusions that served as riding surfaces which exposed this rubber to very serious snow plow damage in service. It is also a demonstrated field problem for any rubber component subjected to heavy traffic as a wearing surface that there is inevitably a pattern of attrition that will occur in the heaviest traffic pattern as can be seen in Figures 10 and 11. Attrition to rubber is particularly noticeable in lower temperatures when the rubber stiffens. Many bridge decks are subjected to rather large

quantities of hard incompressibles and these can wear out the rubber under traffic loading.

Another serious problem was the structural performance of these aluminum systems. Numerous cases were reported of the structural components and spring elements in the system coming loose under repetitive loading. This was either due to inadequate design of the components in the system or the inherent lack of damping under truck loads. It was also reported in the classic New York State field comparison performance test on Collar City Bridge (4) that certain aluminum expansion joints were extremely noisy under traffic loading which means that there was either too much looseness in its fittings or the aluminum itself tended to magnify the noise. The Collar City Bridge is located in a densely populated area and the banging noise from traffic was particularly objectionable.

It is a rarity today to see modular systems incorporating aluminum as structural components for the above reasons. So the lesson to be learned is to be cognizant of its inherent shortcomings as a bridge expansion joint.

Non-Metallic Bridge Joints

By far the great majority of bridge expansion joints are needed for movement requirements of less than 4 inches (100mm). A great many types of aluminum and steel systems have been developed for this market and in the ever present area of intense competition, inevitably, design short cuts have indicated the need for lighter weight, cheaper fastenings, etc. Steel or aluminum beams when used as joint edge members have been particularly noted for their inability to stay in place. Expansion bolts come loose from truck loadings and in some respects are like time bombs. Corrosion of the anchors, fatigue of the welds and the difficulty of bonding something new to old concrete are much in evidence on our interstate highway systems. Added to this is the necessity of preparing shop drawings of the jointing system and the requirement for precise field measurements particularly at the curb and gutter area, a costly and time consuming practice which necessitates different levels of engineering approvals in the process.

One new technique that gives much promise comprises the use of elastomeric concrete to re-build the old joint geometry and the installation of either a high performance field molded or extruded silicone sealing gland, Figures 12a and 12b. The use of silicone tends to eliminate the concern for ultra violet attack so detrimental to most other rubbers primarily because of its inorganic makeup. Cold temperatures down to -40 degrees below zero F (-40 degrees C) do not adversely effect the performance of silicones.

The introduction of non-metallic bridge expansion joints eliminates all this precision drawing and field fitting because the installer merely field molds

elastomeric concrete to the joint interfaces and installs the sealing element from curb to curb. The necessity for lane at a time placement is also simplified with these field molded and field fitted non-metallic systems. As our interstate highway systems get older increasingly requiring more rehabilitation of the joints and decks, the ability to field fit and field mold free of the usual complications required to cut and weld steel components and join standard lengths end to end in a factory is a welcome one.

Modular Systems, Their History and Present Practice - 1963 to Present

As new technology in prestressed concrete lead our bridge designers into longer and longer spans and larger thermal movements, creep-shrink, end rotation and similar movement associated phenomena, there came a growing need for devices to seal movements of 4" (100mm) and over. In 1960 with the introduction of these new compartmented extruded compression seal shapes as compared to field molded sealants, we now had a basic component for modular design. The largest extruded compression seal at that time was a 6" (150mm) uncompressed width element which only provided 2" (50mm) of movement. To extrude something larger than this not only exceeded the practical limits of the rubber extrusion industry, but shapes larger than this would result in joint openings far in excess of the constraints of allowable joint widths on a bridge deck, 4" (100mm) openings being the absolute limit.

Early on, my agrarian roots backgrounded me with an understanding of the mechanism used to prevent cattle crossing into forbidden paths Figure 13 illustrates a device modular in construction, establishing positive control of the distance between the separator beam elements that operates to keep cattle from crossing over and falling through and being injured. It then developed that during the early 1960's the concept of modular bridge expansion joint systems began to evolve, and a patent was applied for, on the concept shown in Figure 14. In order for the compression seal to fit under the I beam separators, we developed a special shape with rounded corners and began to install these systems in earnest throughout North America. Typical of these was a 3 tube system for 4 1/2" (114mm) movements installed on a bridge in Oregon in 1964. We had begun to experience some vertical movement of the separator beams under high speed truck loadings so this sliding hold down device was incorporated in the design (Figure 15).

Concurrently in England going back to 1962, a modular type system was installed on Runcorn Widness Bridge over the Mersey River. Utilizing flat rubber strips bonded to one side only of flat steel separation plates, it attempted to provide a compression seal. Long term creep and displacements plus a very minimal amount of movement per sealing strip element rendered this device impractical from a cost standpoint. This massive system shown in Figure 16 provided only 3 3/4" (95mm) of movement.

In 1962 a German engineer named Otto Fassbinder of the steel and construction company known as Reihnstahl Union Bruckenbau patented and put into practice the RUB System shown in Figure 17.

A single RUB seal element provided plus or minus $2/5"$ (10mm) of movement while a two seal element provided a movement stroke of plus or minus $4/5"$ (20mm). RUB was first installed in 1962 on Frederick Ebert Bridge over the Neckar River at Mannheim in 1962 and saw considerable usage in Germany and Austria. The seal element incorporated side cleats that fit into matching slots in the steel separation beams and were bonded in place. Also, these rubber cleats were eccentricly located so as to create a downward vertical pressure on the separation plates to keep them from vertical movements under traffic loading. Unfortunately, the then unknown, unforeseen tendency of the rubber seal element to undergo yield stress under prolonged tension, compression and vertical tension of the separation plates resulted in the plates becoming very loose in service as these seal elements gradually changed in shape. This compression and tension set of the rubber spelled the demise of the RUB System, although a great many installations had been made throughout Europe. I personally took a license and was responsible for the only RUB installation in North America on a bridge in Ottawa, Ontario in 1967 but its relatively high cost as compared to the patented device in Figure 14 resulted in our going back to the drawing board in an intensive development effort.

We then designed and installed a considerable number of these systems for 4", 6", 8" movements for new bridges in the mid 1960's as the need for larger and larger movements continued to present themselves. We conditioned surveyed the massive expansive joints on the large bridge over the Tagus River in Portugal designed under the direction of Dr. David B. Steinment shown in Figure 18. This structure and approaches involved some 7500 feet (2286m) of continuous bridge. Site engineers advised that close to 100" (2500mm) of movement was involved. Designed to eliminate the maintenance problems occurring on large finger joints, leaf springs were utilized between box shaped steel separation beams to provide equidistance control between these elements. There was however no attempt to seal the joints between the separation beams but this massive expansion joint is unique because it eliminated the problems that would have been involved by using a massive finger joint or a rolling chain type joint that had seen use on major bridges prior to that point in time.

Our first large movement modular system was installed on the Murray McKay First Narrows Bridge in Halifax, Nova Scotia that had a movement requirement of 18" (450mm). Our experimental work included this full scale cyclic modular test machine (Figure 19). We incorporated force measurements, simulated breaking and acceleration loads and after a lot of testing finally decided it was ready for the bridge at Halifax. This bridge designed by Pratley & Dorton was at the time the longest suspension bridge in Canada and Figure 20 shows the

system in service at that time.

We took great pains to design leak proof curb and gutter components since it is in this area of a bridge deck where most of the incompressibles and chlorides find their way to the substructural components of the bridge. Figure 21 illustrates a multi modular arrangement for 9" (225mm) of movement and portrays the care taken at the curb and gutter portion to match exactly the contour of the bridge.

Improvements in the Modular Sealing Concept - 1970 to Present

It was in 1968 that we first came into contact with a German Engineer by the name of Waldemar Koster who had just authored a textbook which examined the origins as well as the present state of the art in Europe of bridge expansion joints (5). In 1969 Herr Koster completed and patented the design of a much improved bridge expansion joint similar in concept to our own system however almost every internal component was superior in design. It was named after the German steel fabrication firm of Maurer. The writer took a license and began to install them in earnest all over North America. The Maurer design incorporated very important controls that eliminated problems we were facing in North America such as (Figure 22a):

1. Mechanically locked seal elements which prevented disengagement of the individual seal elements from their adjacent separation beams during acceleration and breaking loads. No longer would there be a concern about tension and compression set.
2. Counterforce controls for distance control between the separation beams.
3. Sliding bearings underneath the separation beams to considerably ameliorate the fatigue effects of traffic pounding and introduce the first serious attempt for noise control.
4. Steel extrusions which provided corner locking lugs for the seal elements and very rugged separation beams which were in effect double I beams.
5. Separation beams individually welded to the sliding support joists. These presently are being studied for fatigue problems that have arisen with some competing systems where welding was minimized for cost control.
6. Advanced anchorage design which was the result of testing at the Technical Hochschule of Stuttgart. This new design distributed impact

stresses deep into the blockout concrete and paid off well in service.

Our very first installation in North America was on Pine Valley Creek Bridge in California. A 3 tube system for 9" of movement, the curb and gutter detail and prestressment hardware are seen here (Figure 22b).

Another system installed on a bridge over the Ohio River in Kentucky on I-471 is shown in Figure 23. A 6 tube system for 15" (375mm) of movement, the esthetics of equidistance control is quite noticeable.

A very large movement system was installed in 1980 on Pasco Kennewick Bridge across the Columbia River (Figure 24) with a movement requirement of 26" (650mm) designed to take the movement of 2503' (763m) of continuous concrete girder on this epochal cable stayed bridge designed by Arvid Grant.

Still another 26" (650mm) system was installed on Auburn Foresthill Bridge in California which after 20 years of service is still in excellent operating condition (Figure 25).

Of the hundreds of these advanced design modular systems installed throughout North America, Europe and the Far East, very few have given any problems in service. Some have had fabrication problems and some are the victims of design economies and occasional weld fatigue at the junction of the separation beam and support joist however research on this fatigue situation is well under way to sort this out.

The lesson learned here was that the owners should hold manufacturers responsible for their product for a minimum period of time, certainly as much as 5 years. Government regulations seemed to prevent this type of guarantee thinking in specifications and so some irresponsible people began to make design short cuts which got their modular systems into trouble early in their service life. At critical junctures such as the fastening of the support bars to the separation beams, a minimum amount of weld and the type of weld should be specified.

BEARINGS

Elastomeric Bearings

Elastomeric bearings when properly compounded and tested have in general given good performance since their introduction now some 40 years ago. There were some embarrassing failures when some manufacturers, produced sub standard materials but these have now been fairly well eliminated.

They are however not totally free of field problems since they are manufactured as a fixed block of steel plates and rubber and they have no way of knowing

which stage of movement that the bridge deck reflects. The long time creep-shrink shortening movement of prestressed concrete bridges can be close to the thermal movement which means that the bearing may be overstressed and out of phase with its intended movement stroke as shown in Figure 26.

Natural Rubber Verses Neoprene Rubber Bearings

The State of Texas has reported that a great many of their natural rubber elastomeric bearings have a demonstrated field tendency to walk out of their originally placed location under the beams and at the points of bearing. The manufacture of rubber still remains to some degree an art form and the various types of oils, waxes and plasticizer's used in the compounding of rubbers eventually find their way on a time dependent basis to the outer surface of the finished vulcanized part. This migration of components to the surface is known in the trade as "bloom".

Based on a study accomplished by Yura (6) elastomeric bearings were observed in Texas bridges walking out of their original position due to bloom which caused this slipping out of place. Texas DOT, based on this finding, has recently eliminated natural rubber bearings from their specification.

Pot Bearings

It has been reported that the original concept for a pot bearing was the product of the mind of Professor Fritz Leonhardt in 1961 who saw the need for a much improved bearing device to accommodate the longer and longer spans of bridges in the 1960's and 1970's in Germany and Europe. They first saw extensive use in Germany and the writer took a license from the firm of Gutehoffnungshute in 1970 with a view towards extending their use in North America. Post tensioned bridges had yet to see any extensive implementation at this point in time and with the exception of Canada, very few pot bearings were used in USA until much later on, actually toward the end of the 1970's.

The State of Washington and Oregon as well as numerous others have reported having difficulties with pot bearings to the point that some have eliminated their use in favor of spherical bearings. It has similarly been reported that 100% of the pot bearings in the State of Connecticut are experiencing component failures. In fact, to date no manufacturer of pot bearings has been completely free of these failures. To the writer's knowledge, there is no responsible agency at this time in the process of investigating the performance of pot bearings in the USA.

This is not to say that the pot bearings per se are a poor concept however, if it is true that these difficulties are plaguing the industry, it certainly follows that some responsible agency or industry committee should come forward and examine the reported failures. It would appear that an industry committee such

as the American Concrete Institute Committee on Bearing Systems #554 should be charged with the responsibility to make these evaluations and subsequent recommendations for improvements in the state of the art.

Disc Bearings

The Disc Bearing as it is known today, was the product of the mind of former Canadian engineer Edward R. Fyfe. Fyfe had a background in elastomeric chemistry with a degree in chemical engineering and experience for a period of time with the DuPont organization in Canada. The need for an elastomer with structural properties far greater than neoprene or natural rubber became apparent and Fyfe selected a polyether urethane from various existing candidates and conceived the Disc Bearing as it is known today, Figure 27. Considerable testing was performed to verify its reliability structurally and it was determined that a properly compounded polyether urethane disc could be safely loaded to 16 times that of an elastomeric bearing. It had been used previously as a bearing support in the automotive industry and had numerous success stories attesting to its long term reliability under load. Its great design simplicity as compared to manufacturing pot or spherical bearings was attractive to engineers from its onset and so it became a contender for the long span bridge market and has seen successfully used on many structures such as The Pasco Kennewick Bridge in Washington and the Sunshine Skyway Bridge in Florida for up to 4200 kip (18mn) vertical loadings.

There were some difficulties in its early days in getting suppliers to manufacture the disc component to its necessary specification as was the case with early neoprene bearings however, the disc bearing has enjoyed a virtually clear record of ideal performance over the past 20 years. One exception was a failure on a Canadian structure where it was determined that the disc was not manufactured to specification. The disc was replaced and the bearing returned to service. Its flawless performance record in service over the past two decades on so many larger bridges is unprecedented for a bearing concept.

BASE ISOLATION BEARINGS

Definition of Isolation Bearings

It is a departure from the time honored method of brute force treatment of a structure and its support elements wherein the bearings permit a separation of the structure from its supports during earthquake ground shaking and a restoring force to return it to its original position. It is a method of delaying the match up of the period of a given structure and the period of the earthquake thereby eliminating the coincident damaging effect of period matching in a seismic event.

There are now a number of types of isolation bearings which lay claim to accomplishing the isolation design goals of separation and restoration and it is incumbent therefore upon designers to insure that test procedures are developed which sort out claimed technology from actual performance.

History of Isolation

While the patent office is replete with design concepts for isolation bearings which have yet to be tested either on shake tables or in actual quakes, it is only in the recent past that designers have attempted to write them into specifications. The first formal attempt in the USA to write an isolation bearing specification was on a CALTRANS bridge in the early 1970's. Dr. Aristarchos Ikonomu who at that time was Professor of Earthquake Science at the University of Patras in Greece and an accomplished designer of post tensioned bridges that exist in highly active earthquake prone environments conceived and patented the first practical isolation system using shimmed rubber bearings. Dr. Ikonomu while working on a 643' (196m) post tensioned bridge with the additional complication of 40' (12m) radius at one end, saw the need for isolating the support elements from the ground and began to promote the concept of very high shimmed elastomeric bearings to accept the ground motions and a steel fuse which would break at a pre-determined force, the fuse being necessary to maintain structural stability during wind and live loadings (Figure 28).

Recognizing the potential of isolation, in 1973 the writer invited Dr. Ikonomu to come to California for a number of promotional trips including the bridge department of the State of California. After numerous design conferences, a structure was located at Walnut Creek and the isolation system was actually selected for implementation in this particularly hot seismic area. While the project never reached fruition, it was the first isolation specification on a bridge in North America. Later on, base isolation was implemented on a bridge in New Zealand using the lead rubber concept. Subsequent to this, an isolation specification was finally introduced to the USA on a CALTRANS bridge in 1985.

Today there exists quite a number of based isolated bridges in the USA. There also exists quite a large group of firms who are addressing themselves to the isolation concept. Two organizations are hard at work attempting to write specifications. AASHTO Subcommittee on Bridges & Structures, Subcommittee T-3 under Chairmanship of James E. Roberts is working on a specification and an organization known as HITEC is currently developing procedures that will test various candidate systems.

There are always a number of ways to accomplish any design goal like base isolation however for the past 7 - 8 years, a virtual monopoly has existed in the application of these type bearings in USA bridges. The patented lead rubber

bearing concept has been successful in keeping out all other contenders regardless of merit or shake table test verification with such specification phraseology as a history of "implementation of the new system on two bridges for a period of two years". Obviously if this type of specification language were to prevail, there would never ever be any other contenders in the isolation market. As a consequence close to 100 isolation specifications have resulted in only one system being used, the lead rubber bearing. Many design engineers have been duped into thinking that lead rubber is the only viable concept of isolation which assuredly it is not.

Design Questions Concerning Lead Rubber Bearings - Low Temperature Performance

Both the lead and natural rubber which are used in isolation bearings have some interesting performance problems in low temperature environments. Both materials begin to crystallize at lower temperatures which results in stiffening of the rubber block and a glass like resistance to movement of both the lead and the rubber, (e.g. see NCHRP 325). In order for natural rubber to even begin to have the movement for isolation performance at the necessary stiffness criteria, special compounds of rubber have only recently (1995) been resorted to, however the fact remains that many bridges which have been seismically rehabilitated in the past decade with lead bearings admittedly could have serious performance shortcomings. The New England area of the USA which incurs low temperatures with frequencies of minus 20 degrees F (-29 degrees C) has recently had a fairly substantial number of installations of lead rubber bearings which in reality may have questionable ability for seismic performance during winter months.

Landry, Bridge Design Engineer for the State of New Hampshire, in his 1996 report to the Transportation Research Board (7) states that "AASHTO Division II, Section 18.2, Table 18.2.3.1B allows natural rubber elastomeric bearings to have an instantaneous thermal stiffening of not more than four (4) times the stiffness measured at 73 degrees F (23 degrees C) and a low temperature crystallization of not more than four (4) times the stiffness measured at 73 degrees F (23 degrees C). An increase of stiffness by a factor of four (4) decreases the effective period of the bearing by 50%. This decrease in the period moves the structure up the response spectrum, thereby doubling the force transmitted into the substructure. An increase in lateral loading of this magnitude during a seismic event could lead to substructure failure or to a design that would eliminate the use of elastomeric isolation bearings in cold weather regions. Therefore at NHDOT, we believe AASHTO'S guidelines need modification, consequently a test to determine if an isolation system is temperature-dependent is necessary".

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The Snap Back Test

There have been some interesting tests of lead rubber bearings currently under way at the NCEER in Buffalo wherein these bearings which are in use on a structure on a bridge are being subjected to a lateral movement stroke of about 2" (50mm) and then released. The lead rubber bearings have failed to return to their original position (Figure 29) after the lateral force has been removed. What this could be telling us is that the internal shear forces in the rubber are insufficient to overcome the deformation of the lead core.

In early 1996, when ambient temperatures are low, snap back tests were again scheduled to determine low temperature performance of these in service lead rubber bearings. It is difficult to visualize how these lead rubber bearings will actually perform at these lowered temperatures when at 60 - 70 degrees F (16 - 21 degrees C), the stored energy in the rubber bearing is insufficient to overcome the deformation of the lead core.

After the Quake

Based on previously well documented laboratory shake table performance (Figure 30) lead rubber bearings tend to self destruct internally. The lead core tends to heat up and penetrate under the steel shims thereby delaminating them from the rubber in the central portion of the bearing. Since it is a well known undisputed maxim that drilling holes in a bearing can interfere with its optimal structural performance as a load bearing element, the questions arises after a quake, of what to do with in service lead rubber bearings, which are torn apart internally during seismic forces on a shake table(8).

No one knows at this point or has documented at what level and duration of shaking at which these bearings will internally self destruct during a quake.

Sliding isolation systems are relatively easy to inspect and would merely require a simple visual inspection. Movements and forces which have been demonstrated on a shake table at earthquake speeds that may destroy the inner working parts of a lead rubber bearing would have virtually little if any effect on a sliding system.

A Sliding Base Isolation System

Watson and Bradford (9) have developed and patented a concept of sliding isolation bearings which is proving popular with design engineers and is now in service on a number of structures (Figure 31).

Using the standard disc bearing as a base component with a PTFE to stainless steel sliding surface, it incorporates mass energy regulators in the form of dual

direction shock absorber units that are disposed internally inside the bearing block. The system includes all isolation components into the bearing block itself so that there are no installation complications in the field for the contractor. The system is capable of taking earthquake shock loadings from any direction and has been tested successfully on the shake table at the National Center for Engineering Research located on the campus of the State University of New York at Buffalo.

Conclusions

In the aforementioned discussion, the writer has attempted to highlight some lessons learned concerning the various expansion joints and bearings in his experience over the past 50 years. The judgements made, based on testing accomplished at each time and place, resulted in expansion joint and bearing embodiments which were thought to be the very best that could be made based on available data. It has however been a truism that in science and engineering, there is never the very last word. There is only the latest and therein lives the real value of this World Congress on Joints and Bearings where an opportunity is given to reveal the lessons learned to date.



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9. Retrofit of a Medium Span Structure Using a Sliding Isolation System, Proceedings of the Fourth International Conference on Short and Medium Span Bridges held in Halifax, Nova Scotia, Canada, August 8 - 11, 1994, Canadian Society for Civil Engineering, Pages 839 - 847, Paul F. Bradford and Ronald J. Watson.

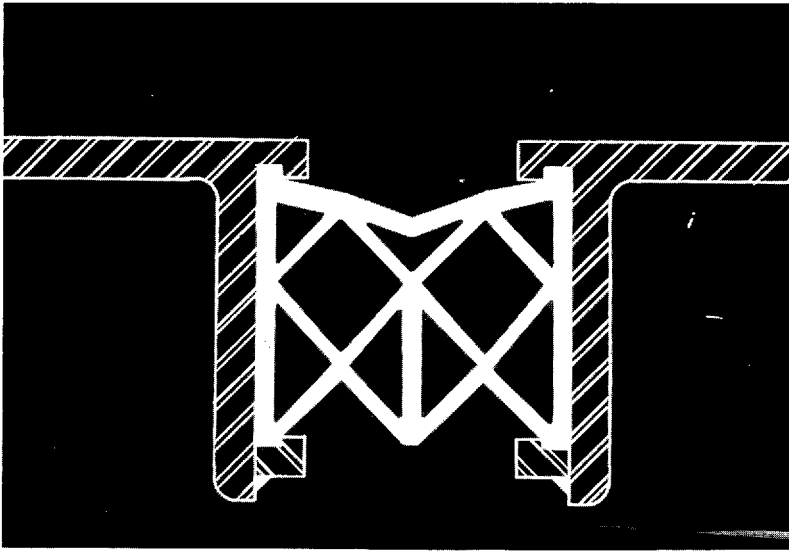


Fig. 1—Early attempts for mechanical locking of bridge compression seal

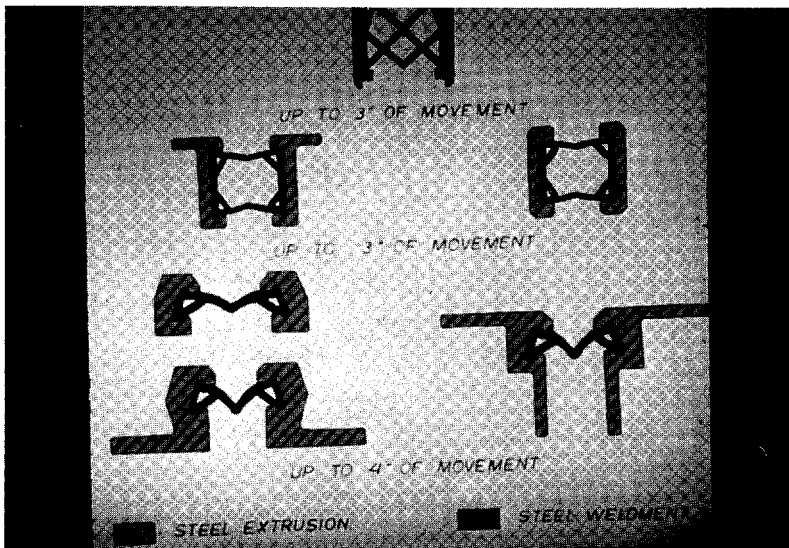


Fig. 2—First examples of steel extrusions for bridge joints



Fig. 3—Transflex rubber cushion system with rubber wearing surface

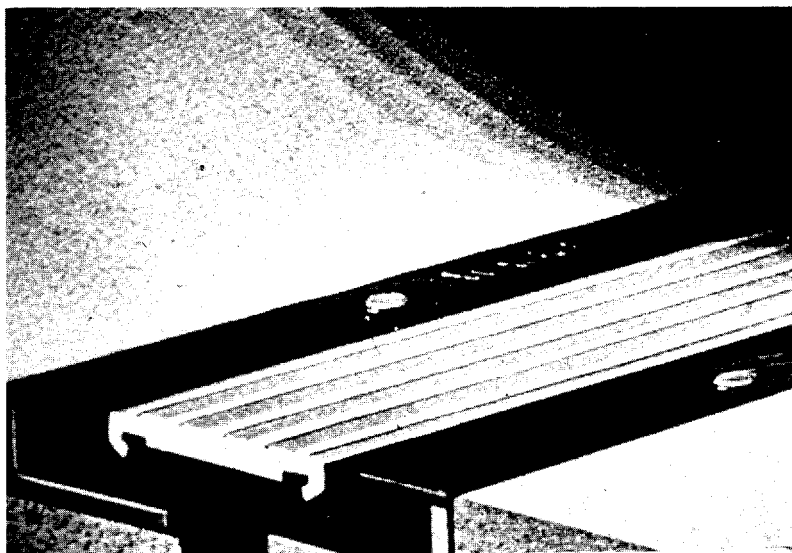


Fig. 4—Waboflex rubber cushion system with aluminum wear plate

Table 1. GENERAL SUMMARY OF ABUTMENT MOVEMENTS				
Movement Type	FREQUENCY		Range In Inches	Average In Inches
	Number of Abutments	Percent Moved		
All Types	439	100.0		
Vertical	379	86.3	0.03 - 50.4	3.7
Horizontal	138	31.4	0.1 - 14.4	2.6
Vertical & Horizontal	77	17.5	0.1 - 50.4	6.9
			0.1 - 14.4	2.2
Two abutments, which raised vertically, are not included in total range or average. Note: 1 Inch = 2.54mm.				

Table 1
General Summary of Abutment Movements.

Table 2. GENERAL SUMMARY OF PIER MOVEMENTS				
Movement Type	FREQUENCY		Range In Inches	Average In Inches
	Number of Piers	Percent Moved		
All Types	269	100.0		
Vertical	234*	87.0	0.03 - 42.0	2.5
Horizontal	52	19.3	0.1 - 20.0	3.3
Vertical & Horizontal	17	6.3	0.3 - 13.7	5.1
			0.6 - 20.0	2.7
*The number of piers with movement included 7 piers which raise vertically. These piers are not included in the total with vertical movement. Note: 1 Inch = 2.54mm.				

Table 2
General Summary of Pier Movements.

Table 3. GENERAL SUMMARY OF DATA ON STRUCTURAL DAMAGE	
TYPE OF STRUCTURAL DAMAGE *	Number of Bridges
Damage to Abutments	69
Damage to Piers	18
Vertical Displacement	45
Horizontal Displacement	68
Distress In The Superstructure	117
Damage to Rails, Curbs, Sidewalks, Parapets	30
Damage to Bearings	34
Poor Riding Quality	12
Not Given / Corrected During Construction	10
None	81

Table 3
General Summary of Data on Structural Damage.

Fig. 5—Abutment and pier movements as reported by Professor Lyle Moulton

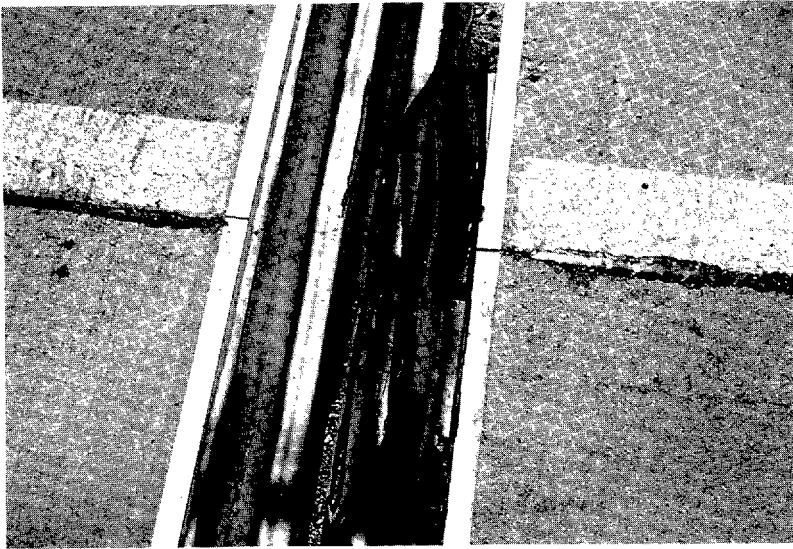


Fig. 6a—Aluminum armor with extruded gland system—Onflex

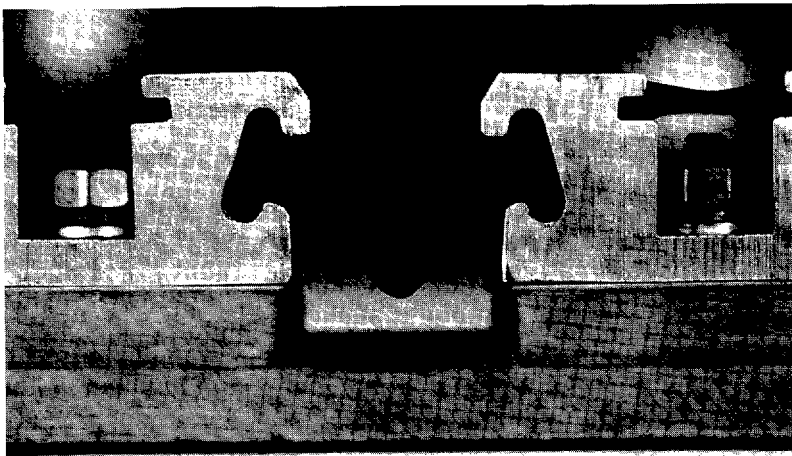


Fig. 6b—Aluminum armor with extruded gland system—Allustrip

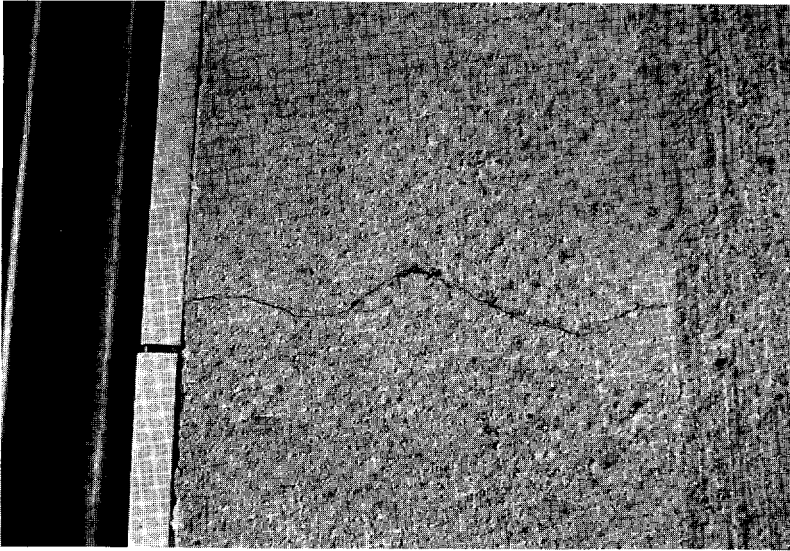


Fig. 7—Crack resulting from difference in expansion coefficient of aluminum and concrete

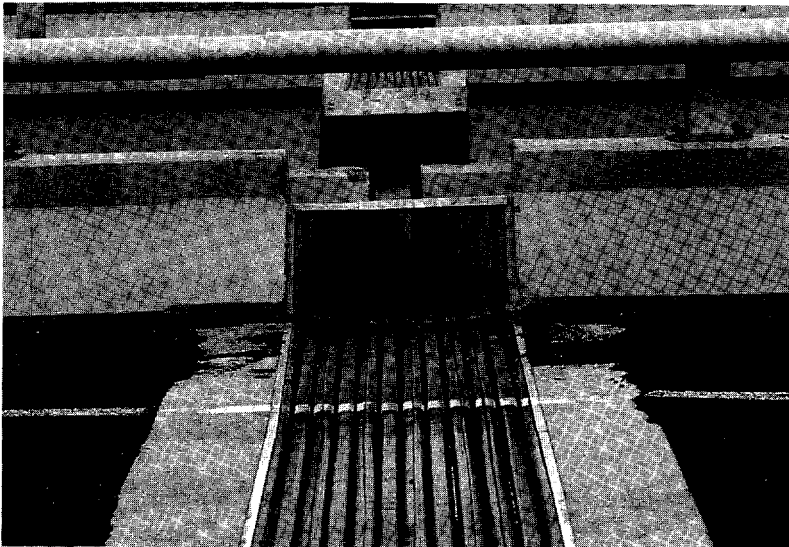


Fig. 8—Aluminum used as armor in large movement modular system—Delastaflex

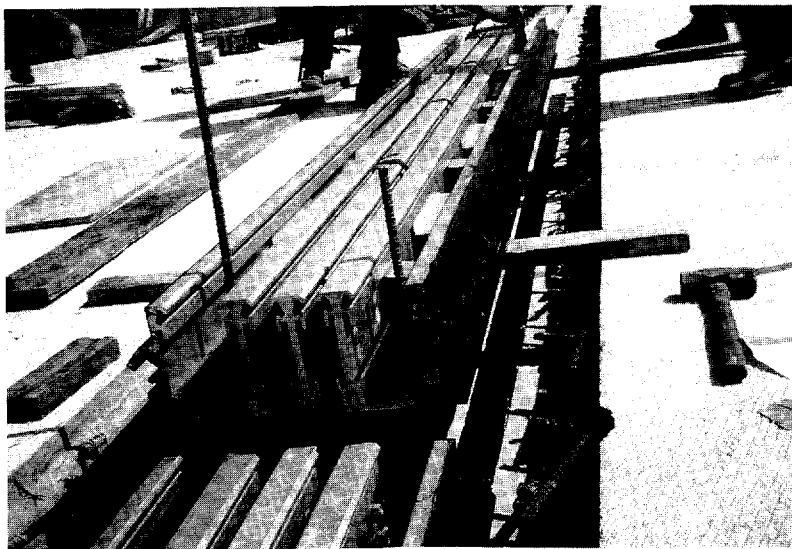


Fig. 9—Aluminum used as armor in large movement modular system—Wabolumn



Fig. 10—Typical attrition to elastomeric wearing surface—Delastaflex

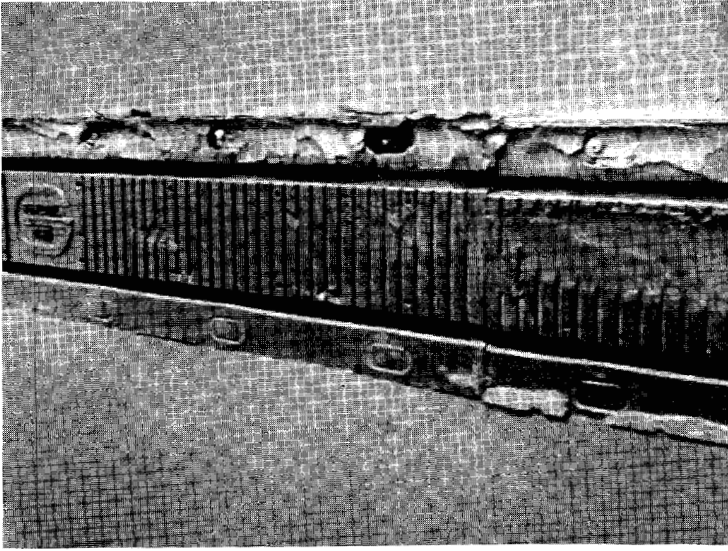


Fig. 11—Typical attrition to elastomeric wearing surface—Transflex

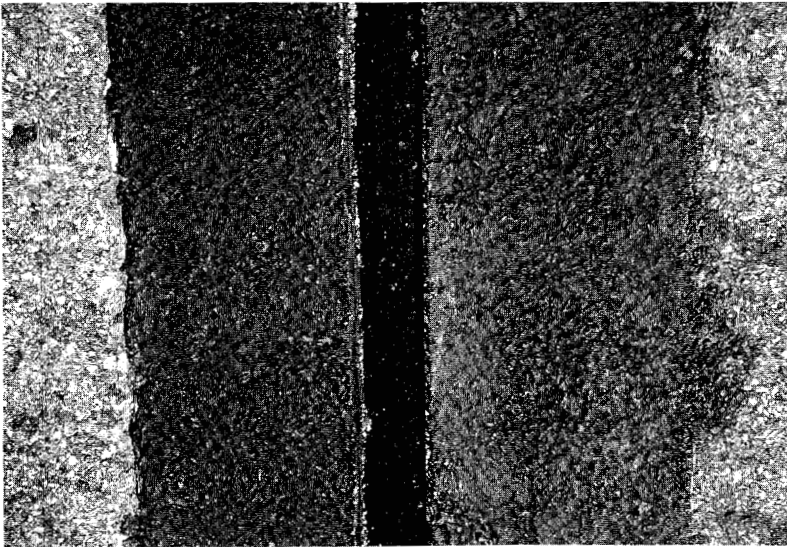
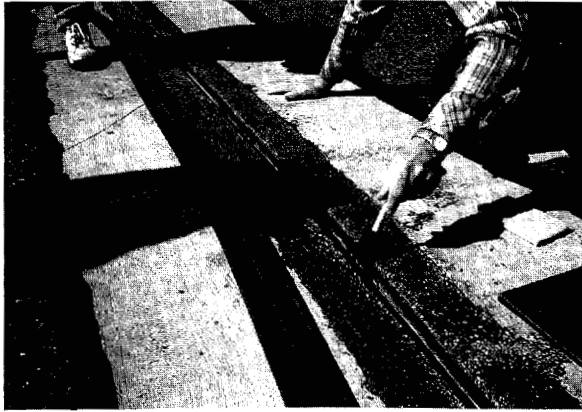


Fig. 12a—Nonmetallic expansion joint with field molded sealant



(a)



(b)



(c)

Fig. 12b—Nonmetallic expansion joint with preformed silicone seal

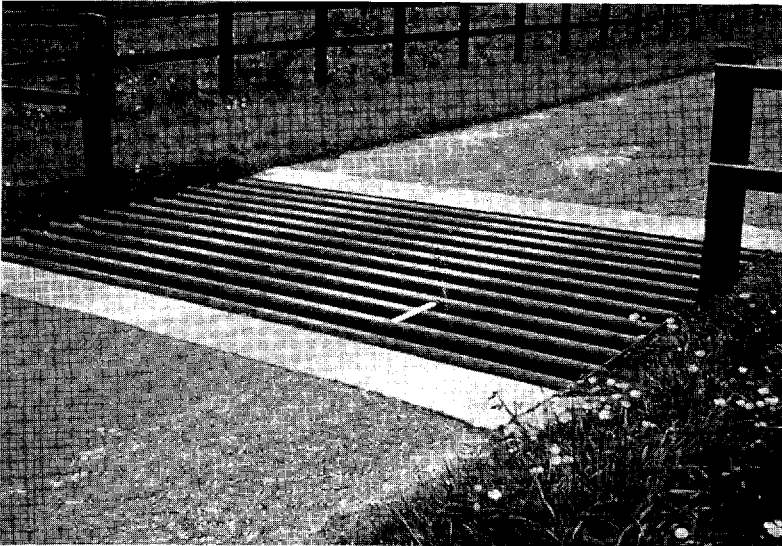


Fig. 13—Cattle guard crossing device

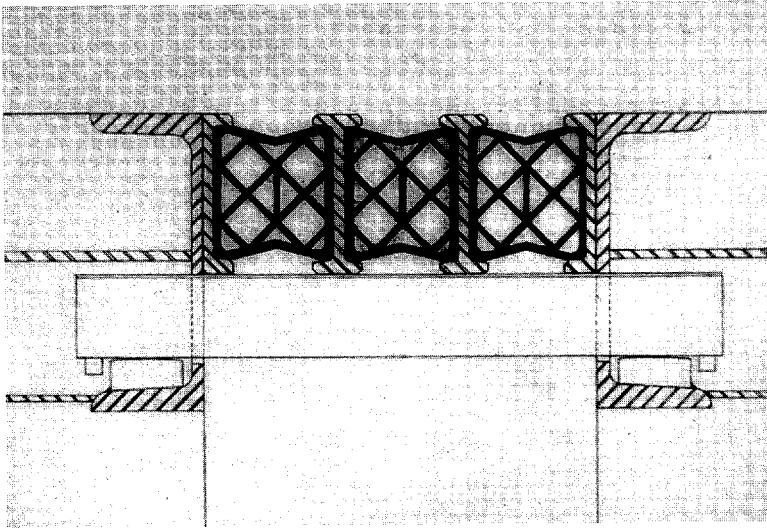


Fig. 14—First patented modular system in North America

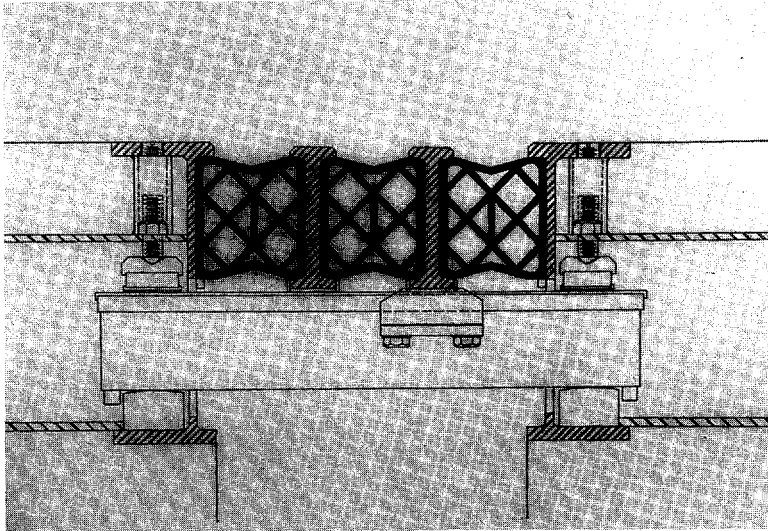


Fig. 15—Hold down device for separation beams

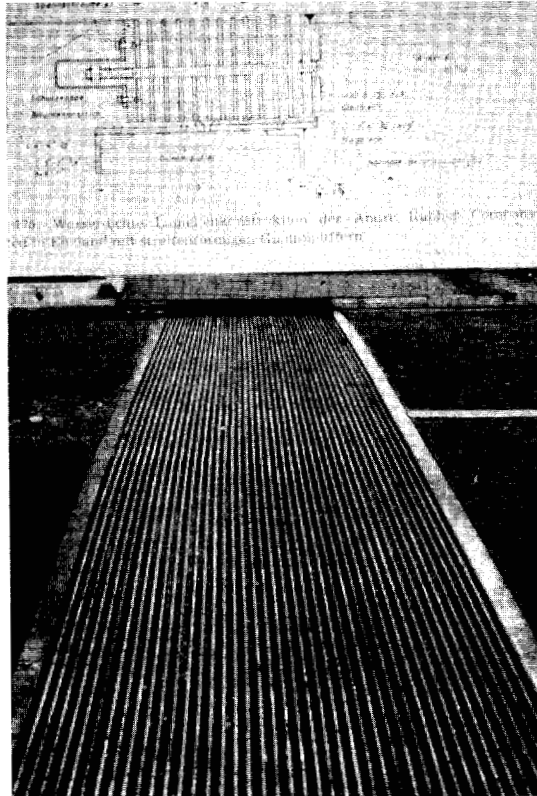
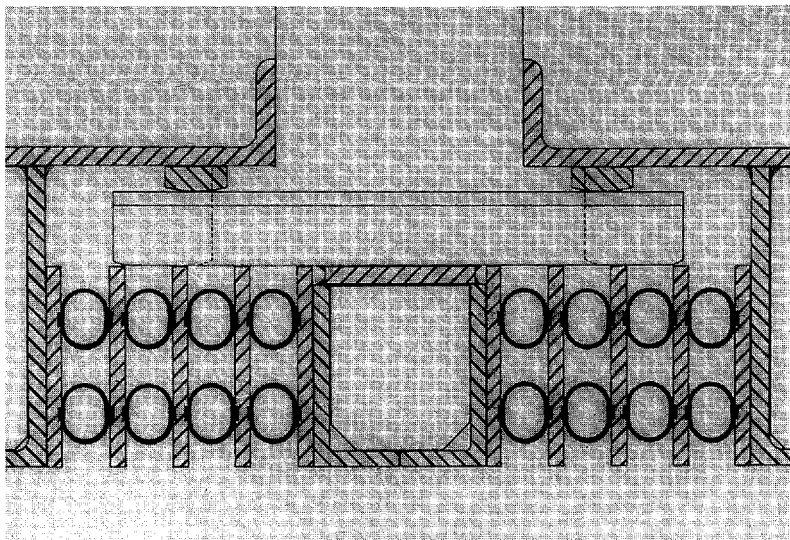
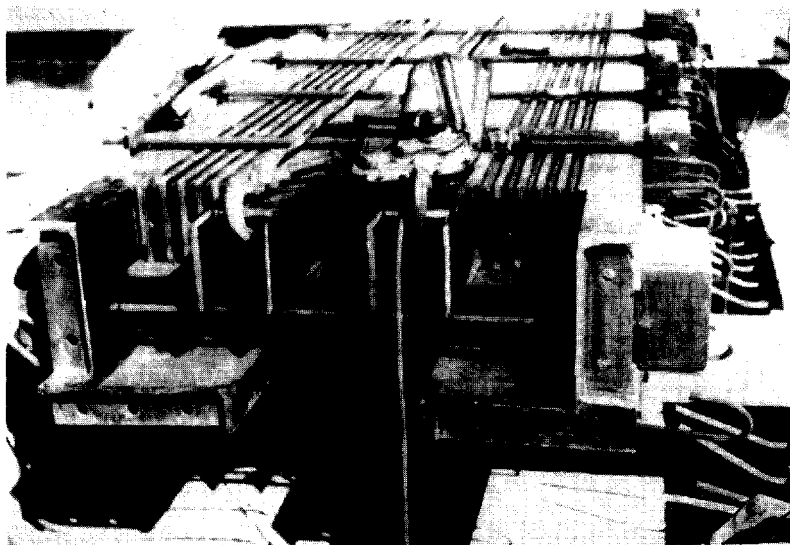


Fig. 16—First modular system—Runcorn Widness Bridge in UK—1962



(a)

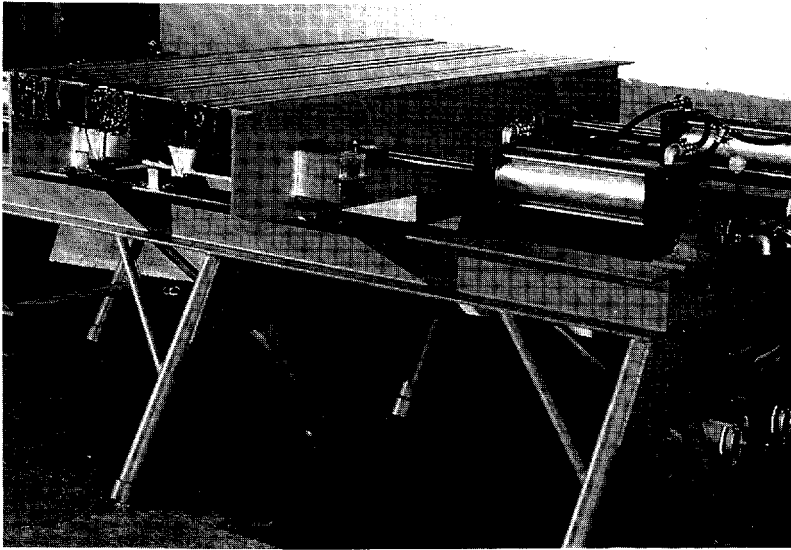


(b)

Fig. 17—RUB system



Fig. 18—Modular system for very large movements—Tagus Bridge in Portugal



(a)



(b)

Fig. 19—Cyclic test machine used to develop large movement modular system

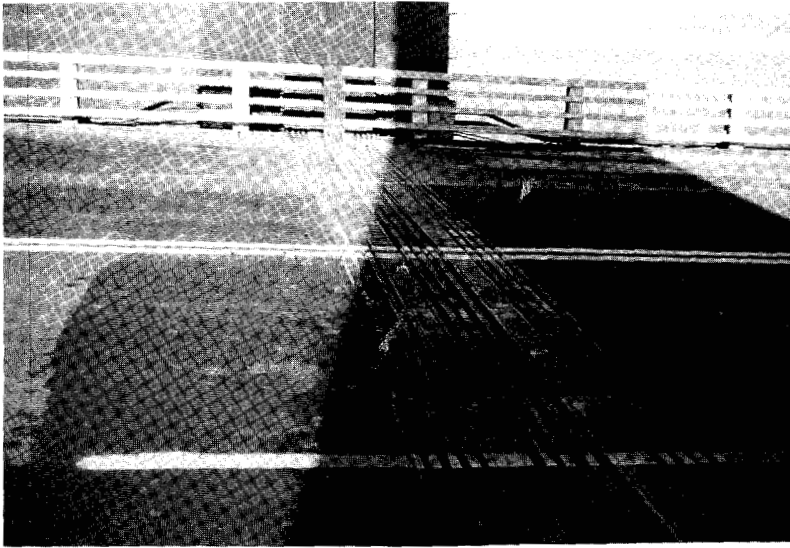


Fig. 20—18-in. (450-mm) movement modular system—Halifax first Narrows Bridge—1968



Fig. 21—Modular system for Papineux Bridge in Montreal contoured to fit curb configuration—1969

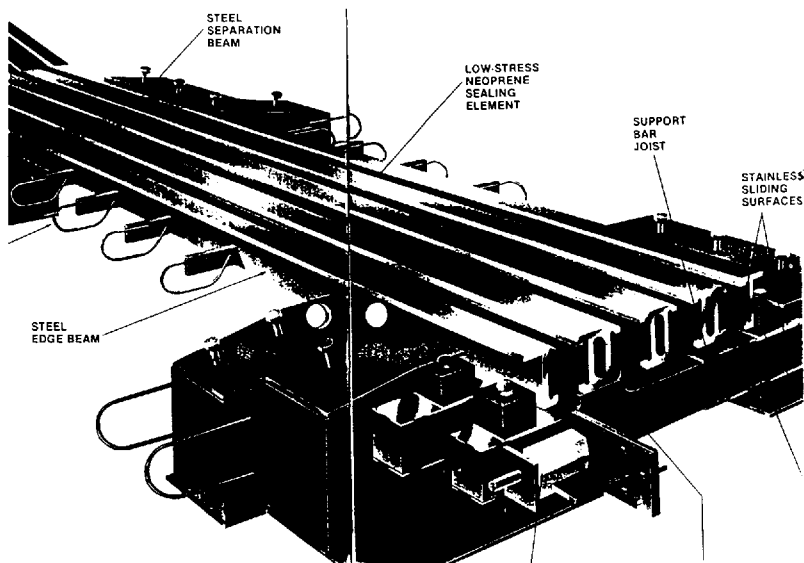


Fig. 22(a)—Maurer system embodiments

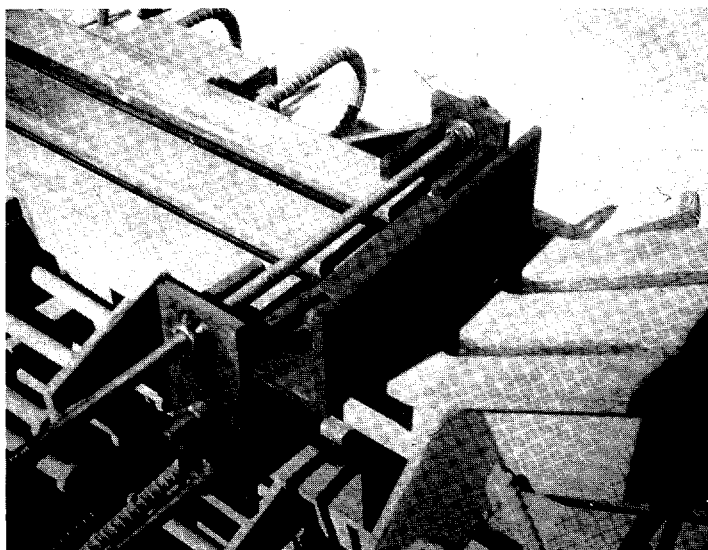


Fig. 22(b)—First installation of Maurer modular system in U.S.A. Pine Valley Creek Bridge in California

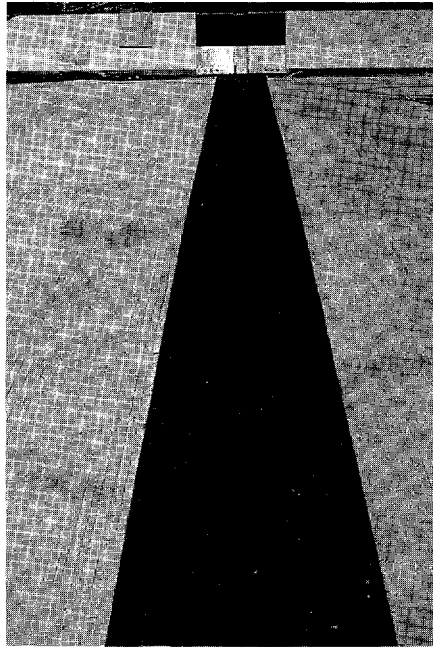


Fig. 23—Maurer modular system on I-471 bridge shows idealized equidistance control between seal elements

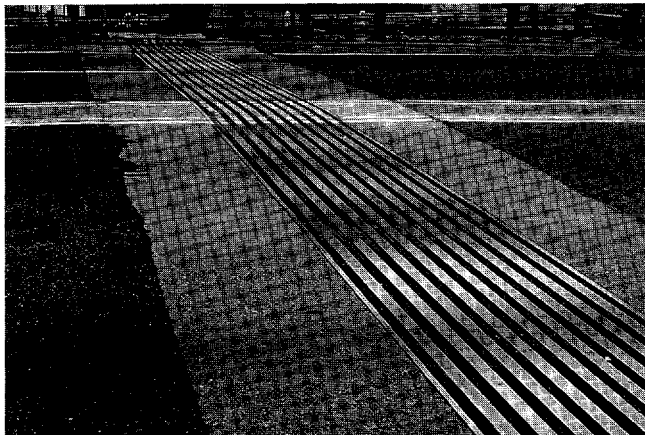


Fig. 24—26-in. (650-mm) movement Maurer system on Pasco Kennewick Bridge over Columbia River in Washington

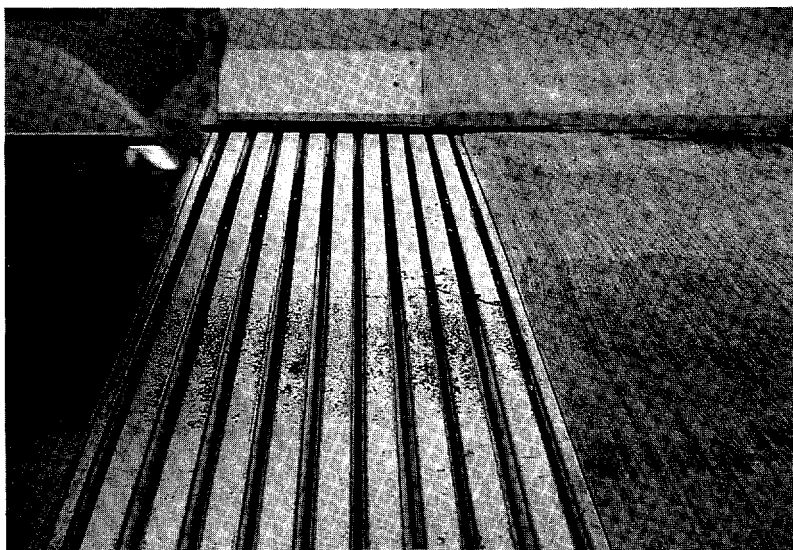
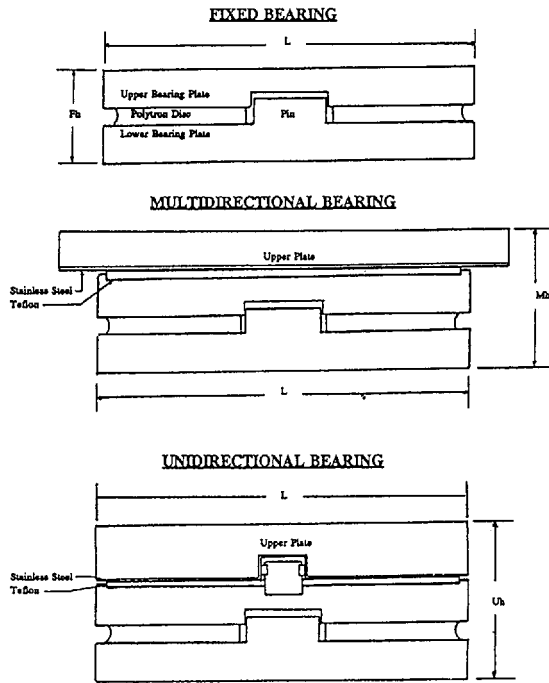


Fig. 25—26-in. (650-mm) movement Maurer system on Auburn Foresthill Bridge in California



Fig. 26—Creep shrink effect on elastomeric bearing requires jacking and repositioning



Standard Design Criteria

- 1 Polytron average compressive stress < 5000 psi.
- 2 Teflon average compressive stress < 3500 psi.
- 3 All teflon shall be recessed and bonded.
- 4 All guide bars shall be recessed and bolted into upper plates.
- 5 Fiber stress in bending < .55Fy.
- 6 Shear stress < .33Fy.
- 7 Horizontal load equals 10% of vertical load.
- 8 Separate sole and masonry plates shall be attached to load plates via recess, bolted, or welded connections.

DIMENSIONS

Model	L	Fh	Mh	Uh
250	10.75	2.2	3.1	4.1
500	14.50	2.9	3.8	5.0
800	18.75	3.2	4.1	5.4
1100	22.50	3.6	4.5	5.9
1500	24.50	4.1	5.0	6.6
2000	28.25	4.4	5.3	7.0

• Model size is the vertical load capacity in kips.
• Dimensions listed in inches.

Fig. 27—Disc bearing

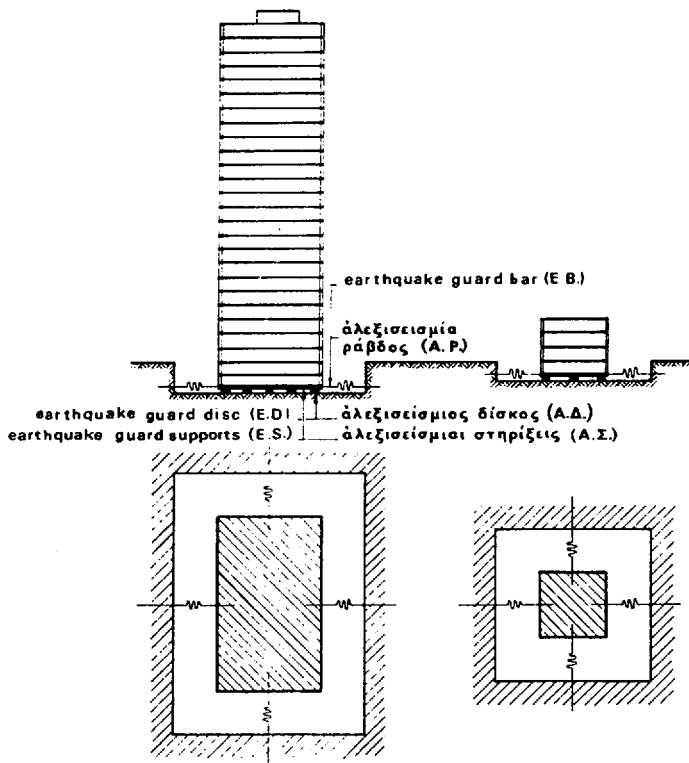


Fig. 28—Shimmed rubber bearing with fuse an early base isolation concept using a rubber stack