

On the Verification of a Finite Element Analysis for Elastomeric Bridge Bearings

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Synopsis: A brief description is given of a recently developed nonlinear analytical model and finite element analysis for elastomeric bridge bearings. The discussion addresses the important problem of determining the stress-strain behavior of the elastomer and adopts a relatively comprehensive nonlinear elastic characterization. The analysis is applicable to arbitrarily loaded bearings of various geometries and accounts for both geometric and material nonlinearities.

The paper discusses the preliminary verification of the analysis as a reliable engineering tool. Such a study involves comparisons of analysis predictions to available experimental results. The comparisons given in the paper are for bearing compression. Such comparisons bring to light problems involving the interpretation of the experimental results due to apparent contradictions among different sources of experimental data. A study is made of possible causes of these differences and how they can be reconciled.

Keywords: bridge bearings; finite element method; plastic, polymers, and resins; structural analysis

2 Element Analysis

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INTRODUCTION

A comprehensive analysis procedure for elastomeric bridge bearings is needed so that engineers can quickly and inexpensively evaluate proposed new designs and novel applications. The alternative of conducting an experimental study usually requires considerable lead time and is relatively expensive. The analysis also can be used to supplement and extend experimental results for the development of design criteria and specifications. Finally, it has proven to be extremely difficult to experimentally measure the internal stress distributions in the individual elastomeric and reinforcement layers. Such information is needed if the results of simple failure studies are to be extrapolated to complex loading states and histories for actual bearings in the field.

The extrapolation of experimental results for elastomeric bearings is made particularly difficult by their inherent nonlinear behavior, which precludes the superposition of the results of simple tests to predict behavior for complex loading histories. The development of realistic design criteria and specifications for bridge bearings requires an understanding and evaluation of this nonlinear behavior.

An analysis for bridge bearings must be capable of predicting, for arbitrary loading histories, the overall response (including possible instability phenomena) of bearings of arbitrary shapes and predict details concerning the local three-dimensional stress and strain states within the rubber and reinforcement layers. Such a broad requirement would suggest the use of a finite element analysis. The analysis must account for the nonlinear effects due to large deformations in the rubber, large displacements of the reinforcement and the nonlinear material behavior of the rubber.

In addition, the time dependent and inelastic behavior of the rubber need to be accounted for. This paper will not address the inelasticity question, instead the elastomeric material will, for the present, be modeled as a nonlinear elastic solid.

There has been a considerable amount of work done on the development of analysis procedures for elastomeric bridge bearings. Reference (1) gives a most complete summary and bibliography of this work. Elastomeric bearings find use in many other situations and have accordingly received considerable additional attention. The reader is referred to reference (2) for a comprehensive summary of these studies for elastomeric bearings for helicopters. References to other works on bearings and related problems can be found in (3-7); the last two papers also reference the previous works of the first author on this and related subjects. For the most part, past investigations have only considered linear behavior and thus are of limited value for most actual bearing problems. Some exceptions are to be found in references (2,8,9).

There are two classical approaches to the finite element analysis of composite systems (10,11). They differ in their underlying descriptions of the layered system, i.e., discrete or composite. In a discrete analysis the detailed inhomogeneous nature of the bearing is modeled so that all material interfaces fall along element boundaries (thus each finite element contains only one type of material). Because of the complicated nature of the layer interaction, in general, very fine grids are required (2,11-13). An alternative to a discrete representation is to model the layered bearing system as an equivalent homogeneous, orthotropic continuum material (3,6,7,10,13,14). An analysis using such a representation will, in this paper, be called a "composite analysis." Because the detailed geometry of the layering is not represented, rather coarse grids may be used (3,6,13,14).

The advantage of a discrete analysis is that the local stress and strain states at and near the layer interfaces, and at the edges of the bearing are directly calculated. The chief disadvantage (often rendering the method entirely impractical) is the excessive computational cost required for the analysis of two- and three-dimensional systems with large numbers of layers, and when nonlinear or inelastic effects, or both, must be included. The advantage of a composite analysis is its low computational cost. The composite approach is used in this investigation; it is a continuation of the work reported in (6,7,13).

In general, the development of an analysis procedure for a structure or a structural component involves four steps. The first is the formulation of a mathematical theory (or specialization or adaptation of an existing theory) capable of modeling the system in question. The second step is the application of an analytical or a numerical analysis procedure for the solution of the theory for specific problems. Thirdly, the predictions of the

4 Element Analysis

analysis are compared to available analytical solutions (when available for simple idealized examples) and experimental results for the purposes of calibrating and verifying the model. Lastly, applications of the analysis are made to relevant engineering problems.

The remainder of this paper contains brief discussions of the developed nonlinear composite theory and accompanying finite element analysis for elastomeric bridge bearings (space limitations will preclude their detailed development), and a somewhat more detailed section concerning some aspects of the verification and calibration of the analysis.

THEORY

There are two fundamental steps in developing the desired composite model for elastomeric bridge bearings. The first is the statement of conventional continuum theory in a form capable of representing all the nonlinear aspects of bearing behavior. The second step involves the homogenization of these equations to yield a composite model for the layered bearing system.

Nonlinear Continuum Theory

Of the three basic sets of equations of mechanics, equilibrium, compatibility and constitutive laws, the first two are well understood and can be written without approximation (assuming that micropolar effects (15) are negligible). The only real choice involves whether one wishes to use the deformed or undeformed body as the reference state.

Because the expressions of equilibrium and compatibility are well established even in the presence of large deformations and deflections, they are only briefly discussed here. An updated Lagrangian coordinate system (16) is used for each element and is defined by the average rotation of the reinforcement layers contained in the element. Thus, the assumption of "moderate rotations" is adequate for modeling the large deflections of the reinforcement layers. However, because the elastomeric layers may experience large deflections relative to the reinforcement layers no approximations are possible in the description of their deformations. The Lagrangian description used for the elastomeric component is expressed in terms of Green strains and Kirchhoff stresses. Before stresses are printed they are converted to the Cauchy (true) stress components. These quantities are all well documented in reference (17) and thus are not discussed further here. It is emphasized that these concepts can be considered to be exact. Unfortunately, the same is not true for the third basic element of solid mechanics, i.e., constitutive equations.

As is the case in many engineering projects, theoretical work on elastomeric bridge bearings involves mathematical manipulations which require material properties. Thus, the successful verification and ultimate use of the proposed analysis depends on the ability to determine the constitutive (stress-strain) properties of the constituent materials (temperature effects are not considered in the present paper).

Much of the current industrial emphasis would appear to be on the use of steel reinforcement whose engineering properties are relatively well known. There is also interest in using materials such as fiberglass and fabrics (18) as possible reinforcement; the relatively difficult question of determining their constitutive properties will not be addressed in this paper (there is, however, a vast literature on the subject, e.g., see (11)).

Unlike steel, with its longer history of engineering use and therefore a better understanding of its material properties, research on rubber is in somewhat of an early stage and often suffers from a lack of common understanding and communication between the producers and engineers. However, for the successful analysis of bridge bearings it is essential that a comprehensive and usable constitutive model be available for the elastomer.

For this study the elastomer will be modeled as a nonlinear elastic material. The neglect of the inelastic effects is usually acceptable, except in cyclic loading cases where material damping is important (19,20). Future work will model the elastomer as a nonlinear viscoelastic solid.

In the following discussion use will be made of the concept of strain invariants. The symbols I_1 , I_2 , and I_3 will be used for these quantities; their precise definitions may be found in most texts on continuum mechanics, e.g., see (17). At this point it is only necessary to note that they are sufficient (in the absence of temperature effects) for expressing the free energy function of a nonlinear elastic body that is isotropic in its initial (undeformed) configuration. The derivatives of the free energy function with respect to the components of strain yield the stress-strain law.

The constitutive model for rubber must be valid for three-dimensional conditions and very large deformations (even though a bearing may be only compressed to a nominal strain of 10 to 12 percent, the local strains in the rubber may exceed 100 percent (9)). The earliest successful three-dimensional characterization for rubber, the "statistical theory of rubber", was developed from a theoretical consideration of the molecular structure of an elastomeric solid. (For the derivation of this theory, and a history of its development, the reader is referred to the classical book by Treloar (19).) The resulting free energy expression is:

6 Element Analysis

$$U = \frac{1}{2} G (I_1 - 3) \quad (1)$$

The material coefficient G is the shear modulus of classical linear elasticity. This expression has two shortcomings. The first is that the predicted nonlinearities are less than measured in actual tests. This deficiency is illustrated, for example, in Figures 5.4 and 5.6 of (19). In viewing these figures it is to be remembered that local strains of magnitudes an order larger than the average bearing strain can occur. The second shortcoming results from the assumption of incompressibility, which was used in the development of the statistical theory for rubber.

In an attempt to improve predictions for large deformations an empirical addition was made to the above expression by Mooney (19), (the results are often referred to as the Mooney-Rivlin model), i.e.,

$$U = C_1 (I_1 - 3) + C_2 (I_2 - 3) \quad (2)$$

where C_1 and C_2 are two material parameters that must be selected, for the particular rubber in question, from a consideration of experimental results. The stress-strain law obtained from this expression is still somewhat deficient in its prediction of material nonlinearity (19) and does not include material compressibility. It is the latter problem that is particularly significant for elastomeric bearings (1,6). Because of the confinement provided by the reinforcement and the large compressive loads applied to bridge bearings, very substantial mean-pressure are developed within the rubber. Thus, even though the bulk modulus of rubber is relatively high, significant volume changes can occur.

In recognition of the need, in certain circumstances, to account for the slight compressibility of rubber and also to improve the nonlinear aspects of the resulting stress-strain law, a more comprehensive expression for the free energy expression has been developed and used in Europe (21):

$$U = \frac{1}{2} B (I_3^{1/2} - 1)^2 + \frac{1}{2} G \left[\left(\frac{I_2}{I_3} - 3 \right) + \mu \left(\frac{I_1}{I_3^{1/3}} - 3 \right)^m \right] \quad (3)$$

The quantities B and G are the classical bulk and shear moduli of linear elasticity, and μ and m are two additional material parameters whose presence strengthens the nonlinearity of the model. It is to be noted that this function includes the previous expressions as special cases ($I_3 = 1$ for assumed incompressible behavior) and yields linear elastic behavior for infinitesimal

strains. The Mooney-Rivlin expression is obtained when $m = 1$, $G\mu/2 = C_1 G/2 = C_2$ and $I_3 = 1$; the typical values suggested in (21) of $m = 2 \rightarrow 3$ and $\mu \approx .01 \rightarrow .05$, however, are quite different than suggested in (19) where values of $C_2 \approx .05$ and $C_1 + C_2 = G$ are reported.

The importance of accounting for the compressibility of the rubber when analyzing elastomeric bearings has also been recognized by Simo and Taylor (22), who have developed an alternative expression for the free energy of rubber.

In this work an approximation of Equation 3 is adopted. In the formulation of the composite theory the fact that the power "m", of the last term, can take on non-integer values causes some difficulties. To avoid this problem the last term is approximated by the first two terms of a Taylor series expansion about a reference value F_0 of the bracketed quantity in question.

$$U = \frac{1}{2} B(I_3^{1/2} - 1)^2 + \frac{G}{2} \left\{ \left(\frac{I_1}{I_3^{2/3}} - 3 \right) + \mu F_0^m \left[\left(\frac{2-m}{F_0} \right) \left(\frac{I_1}{I_3^{1/3}} - 3 \right) + \left(\frac{m-1}{F_0^2} \right) \left(\frac{I_1}{I_3^{1/3}} - 3 \right)^2 \right] \right\} \quad (4)$$

This expression can be simplified and its relationship to Equation (2) clarified by renaming the material parameters:

$$U = \frac{1}{2} B(I_3^{1/2} - 1)^2 + \frac{G}{2} \left[(1 - \mu_1) \left(\frac{I_2}{I_3^{1/3}} - 3 \right) + \mu_1 \left(\frac{I_1}{I_3^{1/3}} - 1 \right) + \mu_2 \left(\frac{I_1}{I_3^{1/3}} - 1 \right)^2 \right] \quad (5)$$

The use, for either predictive or verification purposes, of the resulting rubber stress-strain law in the analysis of a given bearing will require a knowledge of the four material parameters, B , G , μ_1 and μ_2 that describe the free energy function. It would be ideal if their values could be determined independently from simple homogeneous tests on samples of the elastomer in question (such a process is not at all well defined (19)). Unfortunately, this is not possible for bearings that have been tested in the past and is probably out of the question for future commercial bearings; however, it should be a required part of any future research program. Some information concerning rubber pro-

8 Element Analysis

erties; however, must be available (or inferred) before an analysis can be performed.

From a survey of the literature of analytical and experimental work on elastomeric bridge bearings it was found that the kind of material properties necessary for the engineering design of bearings are not commonly considered by the rubber industry when developing and evaluating a new compound (23). The shear modulus, bulk modulus, and the effects of temperature and creep are usually essential for the designer's work, whereas most producers describe their products in terms of chemical compounds, hardness, tensile stress at some elongation, compressive set, etc., and state that their products meet AASHTO material specifications. However, it is generally agreed that AASHTO specifications are only established to control quality and do not lead to information that can be used by either the producer to develop a new material, or the designer for an analytical study (23). In order to achieve a better understanding of rubber behavior, more standard test procedures with meaningful and informative stress-strain results would seem essential.

Attempts have been made to establish relationships between the reported hardness of rubber samples and their engineering properties (i.e., stiffness, or modulus, in compression or shear). If such a correlation were possible it would be extremely useful. Unfortunately, there is no agreement as to its feasibility and/or the number of properties that can be so treated. It is stated in ASTM designations (24) that there is no simple relationship between durometer hardness values and any fundamental property of natural rubber or neoprene. Nevertheless, a number of empirical relationships are to be found in the literature.

For example, Figure 11 of (25) reports experimentally observed effects of temperature on the shear modulus of neoprene for different values of hardness. In Figure 4.8 of (20) the tangent shear modulus is related (experimentally) to durometer hardness values for natural rubber. Figure 1 of (23) compares the experimental curve for shear modulus used by researchers in NCHRP and other curves which were found in the course of their literature survey; substantial differences in the reported values of shear modulus for a given hardness are apparent.

Despite the uncertainties involved in relating hardness to engineering properties, Figure 1 of (26) gives a set of curves which reportedly can be used to determine a number of elastic constants (for natural rubber) from a knowledge of the hardness value. The information of interest for this study is reproduced here in Figure 1. The hardness is described in terms of Φ BS, rather than standard shore A durometer readings. A comparison of Figure 4.8 of (20) and Figure 1 of (26) indicates the possibility of a linear dependence between the two hardness measures; however, it appears that no such relationship has received universal acceptance. For example, it is stated in ASTM (24) that no simple re-

relationship exists between hardness measurements with different instruments. For lack of better evidence it is assumed that they are interchangeable.

An additional difficulty arises when attempting to correlate data from several researchers. That is, natural rubber or neoprene samples with similar hardness values exhibit different behaviors for similar tests. An example of this difficulty was noted by CALTRANS (27) where similar samples were made from 53 hardness neoprene, supplied by different producers. Compressive load-deflection curves of bearing with a shape factor of 3 show difference of up to 4 percent strain for the same stress level (3000 psi). Inasmuch as present AASHTO design specifications recommend limiting the compressive strain to 7 percent, such differences cannot be ignored.

Another important question that enters this discussion is whether or not the bulk (B) and Shear (G) moduli can vary independently for the classes of elastomeric materials used in bridge bearings. If they are in fact independent, then it would most certainly be impossible to correlate both their values to a single measured quantity such as hardness. That is, various combinations of B and G would correspond to the same hardness value and thus could not be uniquely determined by it.

There are two scenarios in which the hardness correlations have value. One possibility is that hardness is at most only a weak function of B and thus could be used to determine G; this possibility is suggested by the approximate theoretical relationship expressing hardness as a function of Young's modulus given in (28); for a nearly-incompressible material Young's modulus is approximately $3G$. The other is, for a given type of rubber, that B bears a fixed functional relationship to G (i.e., $B = B(G)$) that is independent of hardness. If this relationship can be established by some independent means, then hardness would be sufficient to determine both G and B. Although the literature review has produced no evidence to suggest the latter condition, for the present it will be assumed that such a relationship is embedded in the curves for B and G given in Figure 1 (it is further assumed that the relationship is the same for both natural rubber and neoprene). Based on this assumption both B and G can be found from Figure 1 once the hardness is known. If G is known (for example measured in a shear test), it can be entered into Figure 1 to yield a hardness, which in turn can be used to find B. The selection of the parameters μ_1 and μ_2 of Equation 5 will be discussed in a later section.

In addition to the above difficulties, it is generally accepted that hardness measurements are not highly accurate. For example, a material of stated 60 hardness should be considered to be 60 ± 5 in hardness (18), etc. This, as can be seen from Figure 1, results in a significant uncertainty in the shear modulus

10 Element Analysis

value. Further, it should be remembered that a hardness measurement, which is a surface interaction (24), may not necessarily be indicative of averaged properties of the material. For example, the shear modulus of a rubber sample could be significantly different at or near the surface than in the interior. This variation may occur because it is not always possible to obtain the same degree of cure throughout the body during the vulcanization process (23).

In general, the uncertainties in rubber properties made it difficult to quantitatively interpret much of the experimental work on bearings available in the literature. In the section describing model verification the method used in inferring rubber properties will be described.

Composite Theory

The first step in the development of a "composite analysis" is the modeling of the elastomeric bearing as an "equivalent, homogeneous, orthotropic solid". The basis for such a representation involves extending the continuum concept to a level of observation that does not distinguish individual reinforcing layers or individual rubber pads. Classically, the continuum concept has been applied to materials which are inhomogeneous and discontinuous at the microstructure and/or macrostructure levels (e.g., the molecular level for amorphous materials such as glass, crystalline level for metals, aggregate level for concrete, etc.). While the consideration of an elastomeric bearing as homogeneous, as compared to a similar treatment of steel, requires several orders of shift in our scale of observation and thinking, it requires little or no shift for concrete (where the maximum aggregate size is usually in the same range as the thickness of the rubber pads for bridge bearings).

One of the fundamental differences between the modeling of a material such as concrete and an elastomeric bearing as homogeneous is the importance of edge effects for the latter. Edge effects result from the interaction of the structure of the composite with the boundaries of the system. For many composite systems, such as concrete, edge effects are of little consequence; however for others, including elastomeric bearings, they are of major importance (5,6,10,13,14). Thus, in the development of a composite theory for elastomeric bearings special care must be taken to include edge effects; the nature of these effects is described in (3,6,7,13,14).

The development of a nonlinear composite theory for elastomeric bridge bearings parallels the development of the linear theory in (7). Because of space limitations the details of the derivation cannot be presented here, instead only a brief outline

is given and some comments offered on those steps which differ from the linear analysis of (7).

For purposes of this discussion the reinforcing layers are labeled "s" (stiff) and the rubber layers "f" (flexible). To represent the displacement field in an elastomeric bearing two coordinate systems are used, i.e., global and local. The global system is used to model large scale behavior while the local system is used to model small scale phenomena. The global coordinate perpendicular to the layers is denoted by z . The quantities z_f and z_s are local coordinates with origins at the centers of typical "f" and "s" layers respectively. The concept of using "multiple-scales" (i.e., three different vertical coordinates z , z_s and z_f) to represent large and small scale phenomena is borrowed from a branch of mathematics which deals with boundary layer and related phenomena (29).

A set of global field variables (functions of z and the in-plane coordinates x and y) are defined for the bearing. These variables measure quantities that are important and can be observed at the global level. In the eventual finite element analysis their values at the nodes are the global degrees of freedom. In the authors' past work (6,7,14) on the linear behavior of layered systems, the average displacement components over a neighborhood of the system served as a part of this set. However, for the nonlinear analysis the displacements u , v , w of the "s" layer centerline are found to be more convenient. In addition to these displacements, the total set of global variables include the strengths δ_x , δ_y of the two "shear lag edge effects", and the rotations ϕ_x , ϕ_y of the normal to the middle surface of the "s" layer (7). The rotations account for the bending of the reinforcement and the bending edge effects (7). The quantities u , v , w , ϕ_x , and ϕ_y are continuous, smoothly varying functions of the global coordinate z and are defined everywhere over the domain of the bearing. They, however, only have physical meaning for those points which lie on the middle surfaces of the "s" layers, their meanings at other points are defined by interpolation. A similar relationship between δ_x , δ_y and the "f" layers exist.

At this point it is convenient to introduce the concept of the unit cell (representative volume) (6,7,11,14). The unit cell is the smallest fundamental building block of the composite geometry. The unit cell for the elastomeric bearing consists of a single "f" layer and half of an "s" layer on either side, the dimensions in the x,y directions are infinitesimal. As an alternative a single "s" layer and two half "f" layers could be used; the resulting theory would be identical.

The local (at a level of observation commensurate with the layered structure of the bearing) displacement fields for the component layers are expressed in terms of the global field variables (and other intermediate expansion coefficients $c_0, c_1, \dots$) evaluated at the center of the layer in question, and an explicit

12 Element Analysis

dependence on the local coordinate z_s or z_f . (It is to be recalled that the global variables $u, \dots, \phi_y$ are functions of the global coordinate z). Thus, the components of the local displacement field for a particular "f" layer, are expressed as simple polynomials in the local coordinate z_f ; the coefficients in these polynomials are the global quantities (and their derivatives) evaluated at the center of the layer (e.g., c_0^f is the value of the global quantity $c_0(z)$ at the center of the "f" layer in question). These polynomials are of the form $c_0^f + c_1^f z_f$, etc. The explicit forms of these polynomials remain to be determined at which time the relationships between $c_0, c_1, \dots$, etc., and the global variables $u, \dots, \phi_y$ are defined.

The displacement field for an "s" layer is taken from classical plate theory with small deformations and moderate rotations (16) and modified to account for transverse shear and thickness stress induced deformations in the reinforcement (of some importance for elastomeric bearings). Unfortunately, no such comparable, well established theory is available to serve as a guide for the consideration of a flexible layer. However, for the linear case a number of analyses (small deformations, small deflections, and linear elastic material properties) of the behavior of rubber pads are available and are in reasonable agreement, e.g., see (3,6,7,10,30,32). In (7) based on the insight provided by these solutions an approximate local displacement field is selected for the linear analysis. It will be assumed in this work that the same displacement field can serve as an acceptable approximation for nonlinear conditions (of course, the deformations will need to be calculated using proper nonlinear measures, etc.). Subsequent evaluation of the resulting theory indicates that this assumption is justified for the working range of elastomeric bearings, but beyond this leads to an overprediction of bearing stiffness.

The approximate displacement fields for the "s" and "f" layers contain a number of expansion coefficients (e.g., c_0). The enforcement of displacement compatibility at the layer interfaces leads to the expression of all but three of these coefficients in terms of the global field variables $u, \dots, \phi_y$ (7). In the linear case equilibrium considerations were used to determine these last three, however, this step was not actually necessary as the energy minimization process would otherwise approximate these equilibrium conditions. In the nonlinear case the algebra of this step is prohibitive and thus, the three coefficients in question are carried along as additional field variables and are eliminated by "element condensation" using the minimum energy equations.

Once approximations for the local displacement fields for both the "s" and "f" layers have been found all that remains is to use them to calculate the free energy for the system. This involves, of course, the use of the correct large deformation measures of strain (17). For the bearing the free energy can be

expressed as the sum of the energies stored in the N "f" layers and the M "s" layers making up the body.

$$E = \sum_{n=1}^N \iiint_{V_n} U_{f_n} dV + \sum_{m=1}^M \iiint_{V_m} U_{s_m} dV \quad (6)$$

The volume of the nth "f" layer is V_n and the free energy density for the rubber U_f is given by Equation 5, etc. The resulting expression for E contains terms of the form:

$$T = \frac{4}{h_f^2} \iiint_{V_n} (\phi_x^f)^2 z_f^2 dV = \frac{4}{h_f^2} \iint_{A_n} (\phi_x^f)^2 \int_{-h_f/2}^{h_f/2} z_f^2 dz_f dx dy \quad (7)$$

The middle surface area of the layer in question is denoted as A_n and ϕ_x^f is the value of the global field variable ϕ_x at the center of the layer. Explicitly integrating out the dependence on the small scale z_f gives:

$$T = \iint_{A_n} \frac{h_f}{3} (\phi_x^f)^2 dx dy \quad (8)$$

At this point the fundamental approximation of equivalent homogeneous theory is introduced:

$$h_f \iint_{A_n} \frac{1}{3} (\phi_x^f)^2 dx dy \approx \frac{h_f}{h_f + h_s} \iiint_{V_n} \frac{1}{3} (\phi_x)^2 dV \quad (9)$$

The volume of the unit cell (the f layer plus half an "s" layer on either side) containing the layer is denoted as V_n . In the first integral the z (the global variable) dependence of ϕ_n has been evaluated at the center of the unit cell, whereas, in the second it remains free. The approximation of Equation 9 follows naturally from the definition of ϕ_x as a global (large scale) field variable and the fact that the layer thicknesses are small compared to the overall dimensions of the structural component. It expresses the basic equivalent homogeneous concept of viewing the constituents as if they were "uniformly distributed" (smeared) over the body. This process obviously introduces a greater ap-

14 Element Analysis

proximation at a horizontal edge (i.e., parallel to the layers) of the bearing than in the interior. For the last layers at such an edge the unit cell is not complete. The result is that edge effects which arise at such boundaries are not modeled by the present theory. These edge effects are of much less importance than those arising at boundaries which cut across the layers; these latter effects are accurately modeled by the inclusion of the edge effect field variables δ_x , δ_y , ϕ_x and ϕ_y .

The sum of V_n over all "f" layers spans the entire body, i.e., is equal to the total volume V . Thus, the sum of the contributions from all "f" layers yields an integral spanning the bearing; a similar integral is obtained for the "s" layers. These two integrals, when added, give the total free energy for the composite system.

This final expression for energy involves only the global field variables and their dependence on the global coordinates. The integral spans the entire body without any reference to the actual layers making up the bearing. Thus, the desired composite (equivalent homogeneous) theory is obtained. Once the global analysis is carried out for the determination of the global field variables, a return is made to the expressions for the local displacements, strains and stresses in the "f" and "s" layers and the interpolation process described in (6) is used to evaluate these quantities for particular layers in the bearing.

FINITE ELEMENT ANALYSIS

The composite expression for free energy has been used directly in the principle of minimum potential energy for the development of an equivalent homogeneous finite element analysis for elastomeric bearings. The admissibility requirement for the seven primary dependent variables u , v , w , δ_x , δ_y , ϕ_x and ϕ_y is C^0 continuity while the three expansion coefficients that are eliminated at the element level need not be continuous. A 3-D eight node isoparametric element is used with seven unknowns per node. Eight point integration is used for all terms except those arising from the transverse shear strains in the stiff layers and the volume change energy of the rubber layers. Reduced integration is used for these terms to avoid excessive stiffness due to fictitious transverse shear deformations in the "s" layers ("shear lock-up") and due to fictitious local volume changes in the "f" layers (16). The resulting nonlinear problem is solved by dividing the loading history into increments and using Newton-Raphson iteration within each step.

VERIFICATION AND CALIBRATION

There are two possible sources of error in the bearing analysis. The first involves the adequacy of the underlying theory which has been used to model the bearing, and the second the numerical analysis. As noted previously, the one uncertainty in the continuum theory is the characterization of the stress-strain properties of the rubber, including the determination of the material parameters defining the assumed material model. The second question involves the accuracy of the composite finite element model used to approximately solve these equations.

The accuracy of the composite model can be easily checked by comparing predictions to those found from discrete analyses of the underlying continuum theory. For linear two and three-dimensional behavior this has been done and numerous comparisons can be found in references (3,6,7,13,14). For nonlinear conditions results of the present analysis have been compared to the two-dimensional nonlinear discrete results given in (9). All these comparisons have shown excellent agreement in the linear range and in the working range for nonlinear conditions (space limitations preclude giving any of these results here).

In the absence of any direct evidence (e.g. simple homogeneous laboratory tests), the evaluation of the adequacy of the rubber characterization can only be done by comparing predictions of the analysis to experimental measurements made on actual elastomeric bearings. While such comparisons, when successful, have the added advantage of instilling confidence in the overall analysis, when they are unsuccessful it is nearly impossible to deduce what changes are required in the stress-strain law. The verification and calibration phase of this project is still in progress and thus the following comparisons will be restricted to one aspect of bearing behavior, i.e., compression below the buckling load (preliminary predictions of buckling loads have been very successful as have comparisons for compression-shear and compression-bending). The first step is to identify available experimental results. A literature search yielded a number of experimental studies, unfortunately, in many cases they were not sufficiently well documented to permit analyses to be carried out. Five usable studies were found (18,23,26,27,32).

When comparing experimental results from a given source and among different sources, apparent contradictions are noted. Thus, it seems imperative to consider only cases where duplicate tests are available (from more than one source, if possible). Unfortunately, the bearings of each source are, for the most part, of slightly different designs and one has to be content with comparing results for similar but not identical bearings.

The comparison of experimental results from several sources is made difficult because different elastomers (or at least elasto-

16 Element Analysis

mers of differing hardnesses) are used. The shear stiffness of the elastomer is the most important rubber property in determining bearing response (assuming that the bulk modulus is very large). In order to facilitate comparisons, an attempt is made to factor its effect. The finite element program was used to analyze two identical bearings, with the exception of the rubber shear modulus. The comparisons were conducted for two values of the parameter μ_2 . The average bearing stresses were normalized with respect to the shear modulus and are plotted in Figure 2 (values of μ_2 are shown in parentheses). It is seen that over a moderate range of behavior similar normalized bearing responses are found. Thus, it appears that if the bearing stress is normalized with respect to rubber shear modulus, that results from different sources can be meaningfully compared.

It is widely accepted that bearings of the same shape factor but of different shapes yield similar behavior (1). However, there is some experimental and analytical evidence that this is only approximately true (7,18). In an attempt to eliminate this possible source of confusion only bearings with nearly the same shapes will be compared whenever possible.

In all cases the bearings are reinforced with steel and it is assumed that the steel layers are sufficiently stiff that differences in thickness are not important.

When comparing test results for bearings of (nearly) identical designs it is observed that some are initially very soft while others are not. Stanton and Roeder (32) have suggested that this is due to initial imperfections (lack of flatness) in the bearings. In (27) this problem was recognized and in an attempt to rectify the situation the strain that occurred before the bearing load reached 50 psi was ignored. The results in (18) are somewhat clouded by a related problem stemming from an initial curvature of a loading platen. In an attempt to understand this problem, analyses were done of two bearings that were identical except that one had an initial curvature while the other one was perfectly flat. The initial curvature was such that the straightening of it (without compression) would give an apparent strain of 7%. The predicted stress-strain curves are shown by solid lines in Figure 3. The result of shifting the curve for the imperfect bearing 7% to the left (shown by discrete points) is also displayed. This comparison suggests that the effects of initial imperfections can be eliminated by a horizontal shift of the data until the upper portions agree. The data reported in (18,23,27) have all been shifted to some degree, however, the criteria used for the process are somewhat arbitrary. For example the data of (27) was shifted to yield zero strain at 50 psi load; if the solid curves given in Figure 3 were both sifted to yield zero strain at 50 psi stress, erroneous results would be obtained. When the reported curves are averages of several tests, as in (27), the shifting concept is only truly meaningful if for a given stress level the strains were averaged. In general, the horizontal

shifting of experimental data would seem to be justified, however, it only removes the effects of relative initial imperfections and does not indicate the absolute position of the origin.

For purposes of identification of experimental data the following code is used: The letters A,B,C,D,E are assigned to results taken from references (18,23,26,27,32). The letter code is then appended with the number of the figure reporting the results. This is followed by two numbers indicating the shape factor and the ratio of the plan dimensions for a rectangular bearing (for the one circular bearing this last number is replaced by C). For example, D8-6.1-2.5 indicates that the experimental results were taken from Figure 8 of (27), which is here referred to as source D, and that it is a rectangular bearing with a shape factor of 6.1 and a dimension ratio of 2.5. All bearings considered used neoprene with the exception of the one from source C, which is of natural rubber. In Figures 4-14, experimental data is given as discrete points while analytical predictions are reported as solid lines. The notation (a), (b), etc. used in the figures indicate correspondence between identical bearing configurations. The strains taken from (18) were recalculated in terms of rubber (instead of total bearing) thickness so as to have a common definition with the other sources.

Before any analysis can be made values for the four material constants describing the stress-strain behavior of the elastomer, eq. 5, must be selected. For plain strain conditions it is easy to show that the model behavior is independent of the value of μ_1 (19). Even though the behavior of bearings in compression involves three-dimensional stress states, it has been found that the value of μ_1 is of little importance, for compression and thus at the present it is arbitrarily taken to be zero (this corresponds to the use of eq. 3 with $m=2$). A comparison of predictions to the results given in (27) indicates that a value of $\mu_2 = .02$ is reasonable and it is assumed that this value can be used for the other bearings also. The analysis of bearing compression followed by shear in (9) and the experimental measurements for this type of loading (27,32) both indicate that the shear modulus can be determined from the initial shear response in such a test. Thus when such test results were given they were used to select the shear modulus; when they were not available the indicated rubber hardness was used in conjunction with Figure 1. Once the shear modulus was selected by one of the two indicated means it was then used in conjunction with Figure 1 to select a bulk modulus. The shear and bulk moduli (in psi) used in the analyses corresponding to sources A,B,C,E are respectively (100,150000), (94,148000), (112,154000), (80,141000). For Figures 8 and 10 of source D (110,153000) are used, while for Figure 9 (140,163000) are used.

Figure 4 compares unshifted measurements from source E for two nearly identical bearings. In Figure 5 these results have been shifted so that the upper portions of the two experimental curves

18 Element Analysis

are in agreement and match the analytical curve. The close agreement achieved between the two sets of experimental data certainly supports the hypothesis that initial non-flatness of the bearings is responsible for the discrepancies in the initial stiffnesses. If the rubber properties (as given by eq. 5 and the selected values for B, G, μ_1, μ_2) are valid then the matching with the analytical prediction is a way of determining the location of the origin (for a "perfect" bearing).

Figure 6 compares the data for four experiments. The three from source D are for bearings of identical shapes. Those labeled D8 and D10 are from two different suppliers and use rubbers of supposedly the same hardness 50, while that labeled D9 is from the same supplier as that of D8, however the hardness is 60. The experimental data is shifted in Figure 7 and compared to analytical predictions.

Figure 8 displays the shifted data for three square bearings of shape factor 12 along with the analytical prediction. The analytical curve from Figure 8 is repeated in Figure 9 and compared to shifted experimental data and corresponding analytical curves for square and circular bearings, from a different source, of approximately the same shape factor.

In Figure 10 the shifted data and analytical curves for two bearings of approximately the same shape factor but different shapes are compared. In Figures 11, 12 and 13 the behaviors of similar bearings are compared (the ones in 11 are from different sources). Selected results from Figures 5, 11, and 12 for shape factors in the range of 5 to 6 are compared in Figure 14. The shifting in Figures 10 and 11 of the data from source A to the right is only justified if the original shift, resulting from the extrapolation procedure used in (18) to locate the origin, was excessive.

Several analytically predicted stress-strain curves are given in Figure 15 in order to illustrate the effects of rubber modulus and bearing shape on bearing behavior.

DISCUSSION AND CONCLUSIONS

Several preliminary observations can be drawn from the comparisons of the previous section. The completion of this research project will most certainly lead to additional conclusions and possibly some revisions of those stated here. The normalization of the stress by the rubber shear modulus and the horizontal shift of the data is certainly justified when comparing results from the same source (Figures 5,7,8,12,13). The conclusion also appears to hold for data from multiple sources (Figures 7,8,10,11,14), however, further investigation is needed to completely substantiate it. The very stiff small strain behavior of the model and the

experimental results from (18,26) is in sharp contrast to the flexible response reported in (23,27,32). Whether this is completely a consequence of initial imperfections in the latter cases or whether there are also other fundamental differences in the bearings tested is not clear. A factor that is not modeled by the rubber stress-strain law that could possibly account for some or all of these differences is the presence of voids in the rubber for the bearings that are initially quite soft.

If all of the early bearing flexibility can be accounted for by non-flatness then the analytical model would appear to be quite adequate (at least for predicting compressive behavior), otherwise some changes are needed. However, even if the correct explanation has been found, the problem has not been totally solved. The result of shifting the experimental data (or using an analytical curve) is to find the behavior of an "ideal bearing"; of course, the designer must deal with real bearings. Thus, some means for estimating the additional deflection due to imperfections is needed. It should be noted that, in general, the effects of initial imperfections are reduced the more layers in the bearing.

Finally, it appears that the influence of bearing shape on compressive behavior, for a given shape factor, is relatively unimportant (Figures 9,10) in light of the other uncertainties.

Concerning the overall project, it can be unequivocally concluded that the primary goal of developing a more economical nonlinear finite element analysis for elastomeric bridge bearings has been achieved.

ACKNOWLEDGEMENT

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The opinions and conclusions expressed or implied in this paper are those of the authors; they are not necessarily those of the Federal Highway Administration, nor of the California Department of Transportation.

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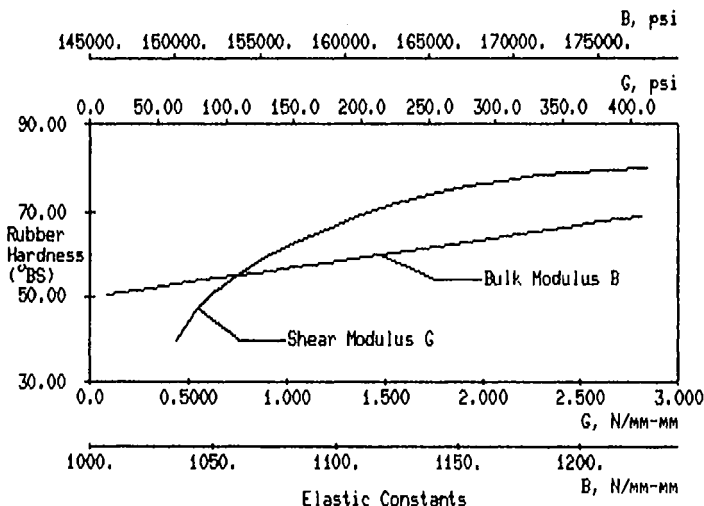


Fig. 1--Correlation of rubber moduli and hardness (26)

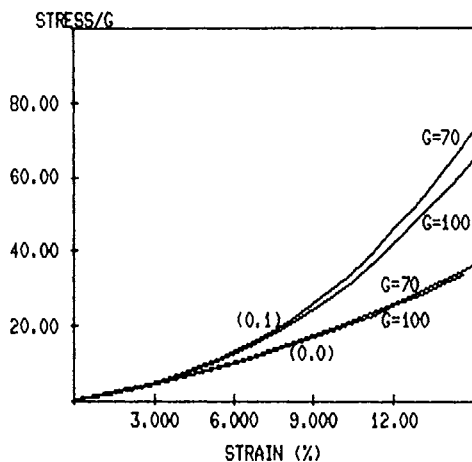


Fig. 2--Bearing stress normalized by rubber modulus

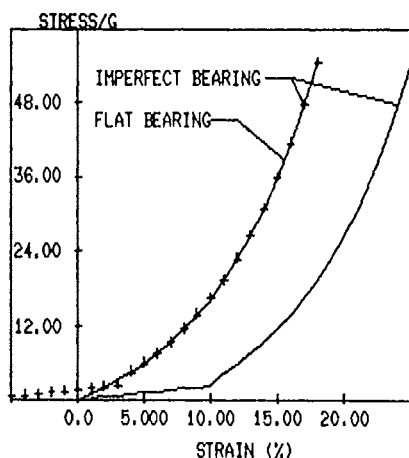


Fig. 3--Effect of initial lack of flatness

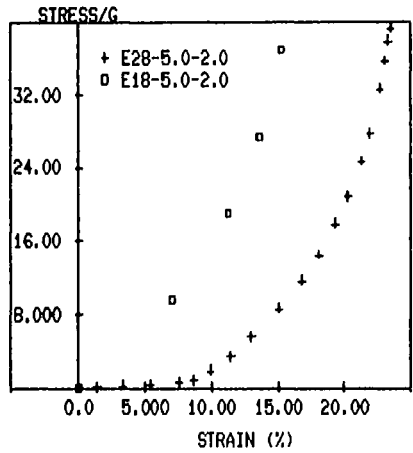


Fig. 4--Unshifted results for "identical" bearings

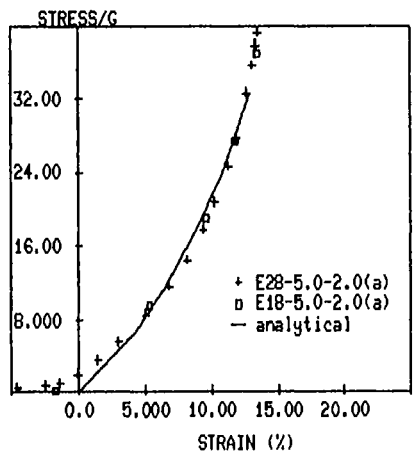


Fig. 5--Shifted results corresponding to Figure 4

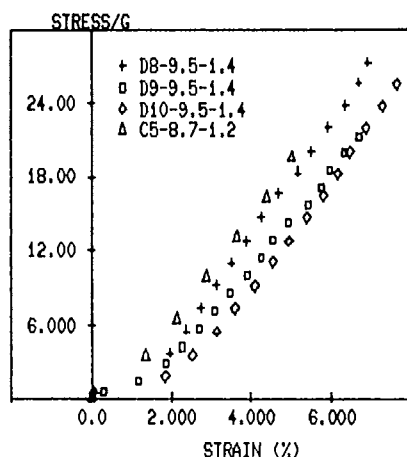


Fig. 6--Unshifted results for S.F. ≈ 9

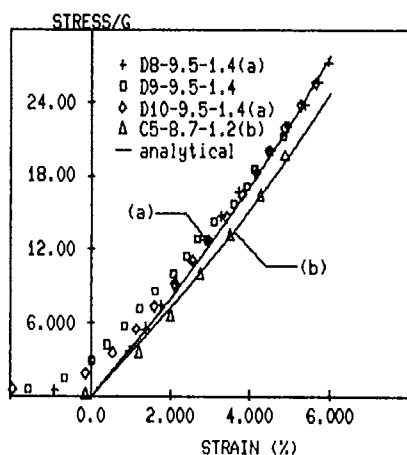


Fig. 7--Shifted results corresponding to Figure 6

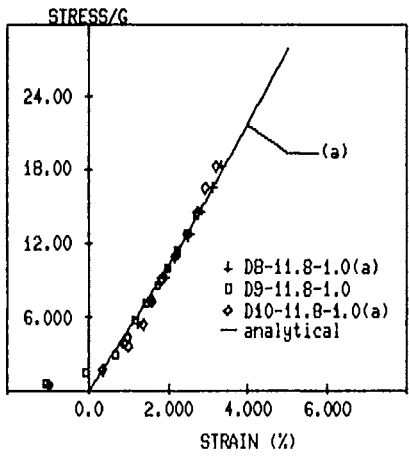


Fig. 8--Results from one source with S.F. $\approx$ 12

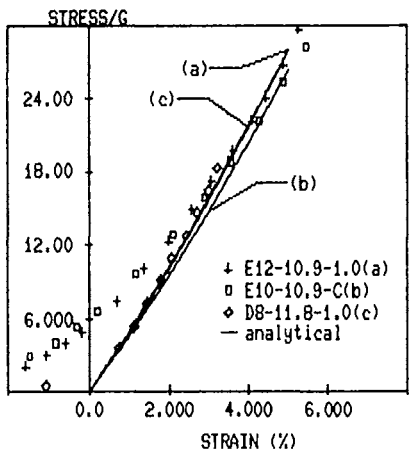


Fig. 9--Comparison of results from (27, 32)

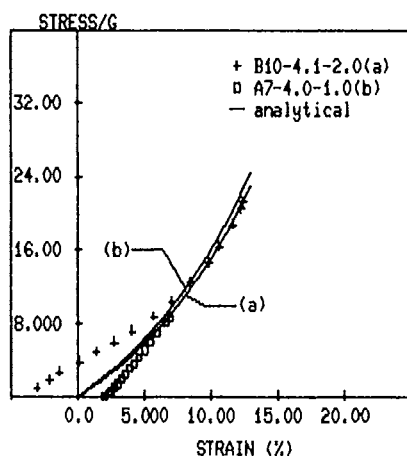


Fig. 10--Comparison of results from (18, 23)

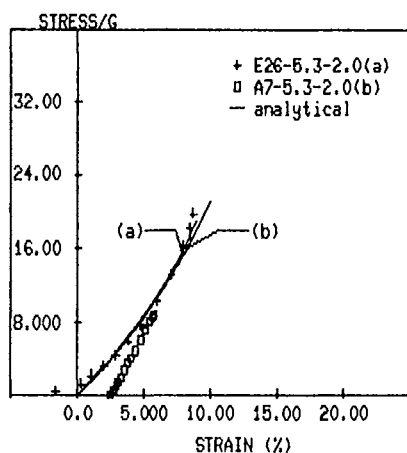


Fig. 11--Comparison of results from (18, 32)

28 Element Analysis

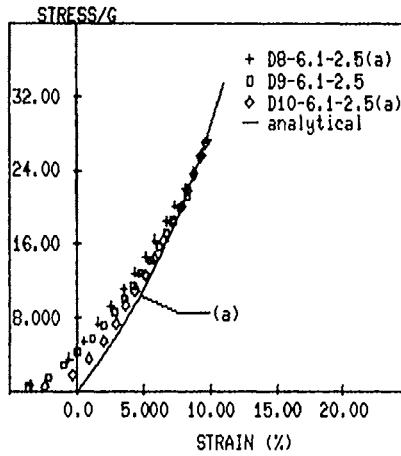


Fig. 12--Results from one source with S.F. ≈ 6

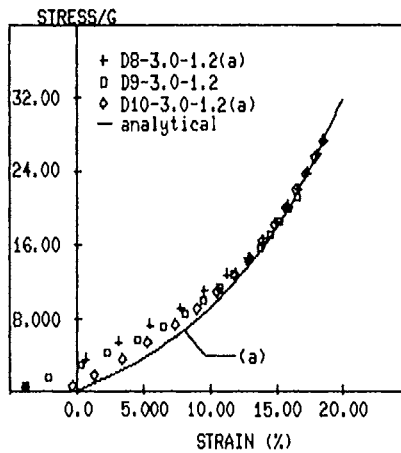


Fig. 13--Results from one source with S.F. ≈ 3

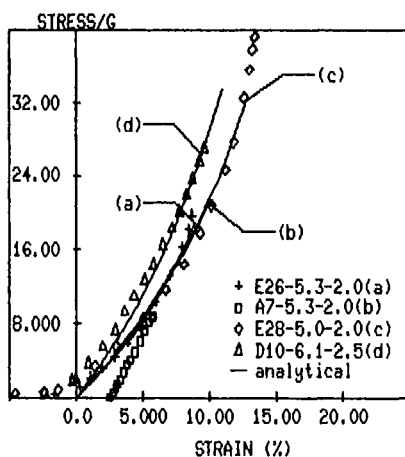


Fig. 14--Comparison of results for S.F. = 5 to 6

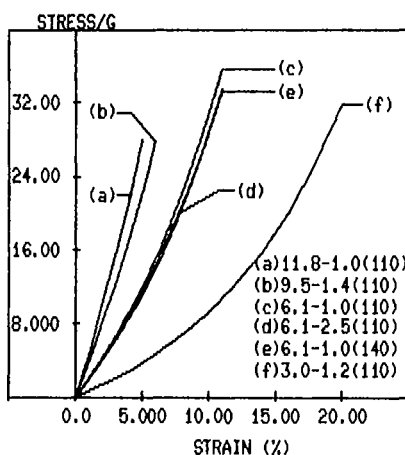


Fig. 15--Analytical predictions

Field Molded Joint Seals in Tension and Compression

by Egons Tons

Synopsis: When joints in pavements, buildings and other constructed facilities are resealed, cast-in-place or "liquid" sealants are frequently used. The sealants are usually placed during warm weather - in Summer - when the joints are at their minimum width. This puts the sealants in tension for the rest of the year.

The paper explores the possibility of sealing joints at intermediate temperatures-in Fall and Spring. Calculations show that the sealant under these circumstances will undergo both tension and compression strains and the effective working range of the sealant may be doubled. This permits much greater freedom when designing joint seals to fit stringent requirements. During the compression cycle the amount of sealant bulging out of the joint must be accounted for if traffic rolls across the joint.

Keywords: compression; joint sealers; joints (junctions); sealing; temperature; tension

32 Field Molded Joint Seals

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INTRODUCTION

During pavement, building, and other structure maintenance operations field molded or "liquid" sealants are most often used. The sealing operations are usually performed during summer months when the weather is warm and relatively dry. It is reasoned that warm weather sealing will be conducive to better quality control and improved bond between the sealant and the prepared joint wall.

If sealant is placed in the joint in warm summer weather the slabs are at their maximum length and the joints are at minimum width. If a sealant with a fully recoverable strain capability is placed in such a joint it will be extended as winter temperatures decrease the slab length and will recover back to its original installation width during the coming summer. Thus the sealant will be essentially in tension all through its life.

Joint seal design procedures for the case where the sealant is subjected to tension only and is physically curving (necking) down into the joint have been published previously (1) (2) (3)*. One of the problems of this case is that the tensile strains in the sealant can be considerable, especially when the sealant depth/width ratio is more than one. The main purpose of this paper is to investigate the case where the sealant is forced to undergo both tension and compression cycles which allows for an increased working range of the sealant.

DESCRIPTION OF THE TENSION-COMPRESSION CASE

Assume that a 1-inch wide rectangular cross section joint is resealed to a depth of 1 inch (depth/width = 1) and flush with the pavement (or wall) surface in October as shown in Figure 1, row 4. The joint opening at that time would be somewhere between the narrow summer width and the wide winter width. As the winter season approaches the joint will widen and the sealant will be extended but not as much as it would for the case with the

*Numbers in parenthesis denote references in the Bibliography.

sealant placed in July. As of average slab temperature rises the joint will narrow down and by March or April the joint may have the same width as it had in October at the time of sealing. If the sealant can fully recover the strain (full memory) the shape of the sealant in March should be 1 x 1 inch and flush with the joint surface. As the temperature climbs higher in May, June, July the joint will narrow down further and the sealant will be compressed. This compression will cause the sealant to bulge or extrude above the slab surface. The maximum bulge and compression will occur at the highest Summer temperature after which the joint will start to widen again as the Fall approaches. Around October the sealant cross section should be 1 x 1 inch again and flush with the slab or wall surface.

From this simplified description it appears that by resealing joints in Fall the sealant is kept in tension for about 1/2 year and put in compression for the other half. If that is true, there should be less total strain in the sealant than in the case where the sealant is placed in the Summer and subjected to tension only.

THEORETICAL CONSIDERATIONS

Previous work has indicated that a sealant with a rectangular cross-section will curve down in the joint as the width of the joint is increased and the sealant is put in tension. Laboratory measurements and calculations show that the upper surface of the sealant has a parabolic shape (1), (2). It is assumed that the tensile strain governs and it is at its maximum along the parabolic top (and bottom) surface (Fig. 1). The strain calculations need good accuracy and are best done on a computer. A simple program is given in Appendix 1.

For this work it was also assumed that when the sealant is put in compression the shape of the bulge will follow a parabolic curve. It was further assumed that tensile strain in the top surface is the critical strain. The height of the bulge above the joint surface is also calculated using parabolic properties.

CALCULATED TENSILE AND COMPRESSIVE STRAINS IN THE SEALANT SURFACE

Taking again the case of 1 x 1 inch joint sealant cross section and subjecting it first to joint expansion forces followed by compression a curve shown in Figure 2 is obtained. The vertical axis indicates the amount of strain in the sealant using the parabolic shape idea for the sealant outer surfaces. The horizontal axis indicates joint expansion (widening) or contraction. At zero percent expansion (or contraction) the sealant cross section is assumed to be square in shape as it was when originally installed.

34 Field Molded Joint Seals

It must be pointed out again that certain assumptions were made before plotting the curve in Figure 2. They are:

1. The curved-down line of the sealant during the joint expansion phase is a parabola. So is the bulge of the sealant when the joint contracts.
2. The curving down and bulging is symmetrical at the top and bottom surfaces of the sealant (Figure 1).
3. The tensile strain along the parabolic surface is uniformly distributed.
4. Only tensile strain along the parabolic surface is considered.
5. All strain imposed during a yearly service cycle is completely recoverable and at zero expansion (or contraction) of the joint the cross-section of the sealant is square (rectangular). The strain "e" in the sealant was calculated using the equation:

$$\% e = \frac{L - W_o}{W_o} \times 100$$

The joint expansion or contraction values " Δ " were obtained from:

$$\% \Delta = \frac{W_x - W_o}{W_o} \times 100$$

where (for both equations):

L = length of the parabolic surface line, bulge or depression (for derivation see Ref.1)

W_o = width of the joint at the time of sealant installation

W_x = width of the joint at any time

The shape of the curve on the right side of the vertical axis in Figure 2 is familiar from publications in the past (1). Here the joint opens up causing tensile strain in the sealant surface. Because the sealant is curving down in the joint the strain values are always ahead of the joint expansion values.

The curve on the left side of the vertical axis shows what happens if the joint contracts right after sealing and puts the sealant in compression. Calculations show that the outer sealant surface is compressed (the strain values drop below the horizontal line) until the joint contraction value reaches about

30 percent. From that point on the sealant outer surface reverses to tension which increases rather rapidly as the joint continues to close. This is a very important practical finding because it allows one to extend significantly the working range of a sealant in a joint. For example, if the original sealed joint width is 1 inch, the seal depth is also 1 inch, and the allowable strain for the sealant is 20 percent, the maximum allowable movement for such a joint would be about 18% or .18 inches if it is sealed in Summer and only joint expansion effect is present. If the same sealant is placed in the same joint at some predetermined intermediate time so that the joint is allowed to contract from its original sealed width this working range can be increased to $.18 + .39 = .57$ inches or about 3 times more (see Figure 2). As it will be seen later, not all this working capacity can be always used because the height of the bulge has to be watched, especially in pavements.

Figure 3 is an extension of similar calculations and resulting strain curves for sealant cross-sections with different shape factors or depth/width ratios. Here the original sealed width of the joint is held equal to 1 ($W_o = 1$) and the sealed depth is varied. As can be observed, the smaller the depth of the sealant the lower the tensile strains for a given joint expansion or contraction. The sealant develops first compression strain as the joint contracts and then goes into tension. This compression phase is negligible for $D/W_o = 2.6$ but increases steadily as the D/W_o ratio decreases. It is understood that a joint seal 1 inch deep and .2 inches wide is difficult to produce and control in the field ($D/W_o = .2$). However even a 1 inch wide and 2 inches deep seal ($D/W_o = 2$) designed for 20% maximum strain can take about twice as large joint movement if both tension and compression cycles are utilized (the sealant is put in the joint in Spring or Fall). Again, parallel to the strain values the bulging and depression of the sealant in and out of the joint must be considered.

LIMITS FOR BULGE AND DEPRESSION OF THE SEALANT

If a sealant is placed flush with the surface of the joint and there is no traffic passing over the bulge there may not be a problem. Such may be the case in many building and other joints. On the other hand, sealant protruding upwards from a highway joint will be touched by traffic and distorted. This can affect the strain recovery possibilities of the sealant as it goes through the seasonal joint expansion and contraction cycles. It may be possible not to seal such joints flush with the slab surface but place the sealant some distance below so that some bulging can be accommodated.

Figure 4 shows curves for H values for the case where $W_o = 1$ and different D/W_o ratios. The value H is the distance between the highest and the lowest point on the sealant surface

36 Field Molded Joint Seals

be calculated using simple parabolic relationships (can also be found in Ref. 1). There are no set norms for bulging or depression values. As an example .25 inch bulge or depression could be set as a maximum for a highway joint. If such a limit is assumed the maximum allowable joint expansion and contraction for a given D/Wo ratio can be obtained from Figure 4. As it can be seen the slope of the curves is steeper in the joint contraction cycle as compared to the expansion case. A sealant with 1 x 1 inch cross-section will bulge up .25 inches at about .25 inch joint contraction while it takes nearly .50 inches of expansion to reach the same magnitude in depression. Thus the curve-down values may not be as critical as the strain magnitudes for the joint expansion case. On the other hand, the bulge height values may be more important than strain in the joint contraction case.

COMBINING STRAIN AND BULGING OR DEPRESSION LIMITS

Figure 5 shows both the strain and the H (bulge or depression) curves plotted together for 1 x 1 inch original cross-section joint. If maximum permissible strain in a sealant is assumed to be 20% (.2 inches) the joint can expand about .18 inches. The H curve allows at least .5 inches expansion. During the joint contraction cycle the H curve permits .25 inch contraction while the strain criterion gives about .4 inches. Thus in the joint expansion cycle the strain governs while in the contraction cycle the bulge (H) sets the limit. By using both cycles the maximum permissible joint movement is $.18 + .25 = .43$ inches instead of .18 inches if only the joint expansion cycle is used. Even in the case of $D/W_o = 2$ the total allowable joint expansion is about .27 or twice as much as when using the tension cycle alone. In other words, if a sealant is made to work both in tension and compression during its yearly service cycle the allowable joint movement can be increased considerably, maybe doubled.

PRACTICAL APPLICATION

There are various ways to approach the design of a joint seal. Whatever the design approach, the number of variables which affect the sealant performance in the joint on a given construction job should be the same. The most important ones may include:

1. Concrete slab length, L.
2. Joint width at the time of sealing, W_o .
3. The depth to which the joint is sealed, D.

4. Design slab movement during a yearly cycle, dW .
 - a) due to temperature changes in the slabs (Fig. 6).
 - b) due to moisture changes in the slabs.
 - c) restrictions to movement due to subbase friction, frozen dowels, etc.
5. The maximum allowable tensile strain in the sealant being used.
6. The maximum allowable bulge or curve-down of the sealant in the joint.
7. Other important and less important properties of the sealant material, the concrete, climate, geometry etc. which could be considered.

The above variables are listed for use in the following calculations. Using the ideas and discussion developed in this paper the first six variables can be manipulated and coordinated numerically to get some practical answers, such as, what should be the width of the joint at the time of sealing if the slab length, temperature changes, etc. are given. It will be seen that the factors 1 to 6 in the above listing can be assigned numerical values to answer the question: How wide? To obtain a numerical answer to the question above some curves and charts would be helpful. Further work is needed to develop them. For the time being joint seal design examples will be given using Figures 3, 4, 6 plus some calculations.

SAMPLE CALCULATIONS

Assume that it is desired to calculate the needed joint width at the time of sealing ($W_o = ?$).

Assume that the other information given is as follows:

1. Concrete slab length is 30 feet ($L = 360$ inches).
2. Joint width at the time of sealing-to be calculated ($W_o = ?$).
3. The depth to which the joint is sealed is .75 inches ($D = .75$).
4. Design slab movement during a yearly cycle is .24 inches ($dW = .24$). This can be obtained in different ways. Some agencies may have field measurements available. It should be mentioned that a number of joint movements must be measured, average and standard deviation should be obtained and some confidence limit value for the joint

38 Field Molded Joint Seals

movement should be set. The value above (.24 inches) was obtained using Figure 6 from Reference 4 which is based on 95% confidence limit for joint movement, coefficient of variation of 30%, and thermal coefficient of expansion for the concrete assumed to be 5.5×10^{-6} . Only temperature effects on the slab were considered and ΔT was assumed to be 80 F (difference between highest and lowest slab temperature). Effects of moisture and friction were ignored.

5. The maximum allowable tensile strain in the sealant is assumed to be 20%.
6. The maximum allowable bulge of the sealant out of the joint during compression and curve-in during the tension cycle taken as .25 inches (the sealant top surface will be recessed .25 inches below the slab surface to accommodate the bulge).
7. Other factors not considered.

A. Solution by trial-and-error, first calculation

- a) Assume that $W_o = 1$ inch; then $D/W_o = .75$ (3/4).
- b) From Figure 3 at the 20% maximum allowable strain in the tension mode the sealant can take about 18% joint expansion or .18 inches (the value to remember is .18 inches).
- c) From Figure 4 at 18% joint expansion the depression value $H = .1$ inches which is less than the allowable .25 inches. So the strain in b will govern.
- d) From Figure 3 the $D/W_o = .75$ curve on the left side crosses the 20% maximum allowable strain line for the sealant at about 47% contraction of the joint from its original one inch sealed width.
- e) From Figure 4 only about 30% or .3 inches contraction of the joint is permitted so that the bulge H does not exceed the maximum allowed. The H value will govern (the number to remember is .30 inches).
- f) The total joint width change (expansion plus contraction) which the sealant can take and not exceed the tensile strain limit and the bulge H maximum value is $.18 + .30 = .48$ inches.
- g) The predicted joint movement from item 4 above was .24 inches, which is only 1/2 of what the seal is capable.
- h) Next $W_o = 1/2$ will be tried.



B. Second trial-error calculation

- a) Assume that $W_o = 1/2$ inch: then $D/W_o = 1.5$ (3/2).
- b) From Figure 3 the sealant can take about 15% joint expansion or $.15 \times .5 = .075$ inches (the value to remember is .075 inches).
- c) From Figure 4 the value for $H = .13$, which is small.
- d) From Figure 3 the $D/W_o = 1.5$ curve crosses the 20% strain limit at about 28% joint contraction.
- e) From Figure 4 only about 18% or $.18 \times .5 = .090$ inches is allowed to keep the bulge below the .25 limit (the number to remember is .090 inches).
- f) The $W_o = .5$ inches sealant can take $.075 + .090 = .165$ inches joint movement. Since the estimated joint design movement is .24 inches in Item 4, the initial joint width should be larger than 1/2 inch.

C. Comparing first and second trials

During the first trial calculations the W_o was assumed to be 1 inch and the calculated total allowable joint movement in tension and compression cycles was .48 inches. During the second calculation the W_o was assumed to be .5 inches and the calculated allowable joint movement came out to be .165 inches. It is apparent that the joint width as installed should be somewhere between .5 and 1 inch. Assuming a straight line relationship, it is estimated that W_o should be between .6 and .7 inches for this project.

D. Final check

- a) $W_o = .65$, $D/W_o = 1.15$.
- b) Figure 3, expansion movement is 17% or $.17 \times .65 = .11$ inches.
- c) Figure 4, curve-in value is about .11 inches (small).
- d) Figure 3, contraction movement is about 37% or $.37 \times .65 = .24$ inches.
- e) Figure 4, allowable joint opening due to the .25 inch bulge is about $.22 \times .65 = .14$ inches.
- f) The sealant can take $.11 + .14 = .25$ inches which satisfies the .24 inch design requirement. Use $W_o = .65$ inches.

40 Field Molded Joint Seals

E. Field considerations

In this example it was assumed that the average temperature of the concrete slabs will differ by 80 F between the Summer and Winter seasons. Since sealant is supposed to take .14 inches of the joint movement in compression and .11 inches in tension the sealing time should be arranged somewhere near the middle between the high and low temperatures of the slabs. For instance, if on a given job site the slab temperatures are minimum 15 F in the cold season and 95 F during the warm season the sealant should be placed in the joint at around 55 F (the average temperature of the concrete slab). A more accurate calculation is: $15 + (80/.25) \times .11 = 50$ F. This indicates that in order to utilize the full tension-compression advantage the sealant will have to be placed in cooler weather in Fall or Spring, at around 50 F.

Practical considerations may dictate the width of the original joint to be $3/4$ (.75) inches. In that case there would be an additional safety factor built in and even the sealing temperature could go up slightly.

F. Other calculation choices

In the previous example the required width of the joint at the time of sealing was calculated using given data. In a similar manner the concrete slab length or the sealant depth D, or the bulging H could have been used as the dependent variables.

GENERAL DISCUSSION

This brief presentation has dealt with the utilization of liquid type sealants in compression and tension. The use of the compression cycle is helped greatly by the discovery that the outer-most layers of the sealant plug first are subjected to compression strain followed by zero strain and then going into tensile strain. This extends the possible working range of the sealant very significantly. For example in the calculated example above the compression cycle was able to accommodate more than half of the estimated .24 inch joint movement. If the bulge limit could be set above .25 inches additional joint closing could be allowed before the tensile strain limit of 20% is reached. Higher limits on the bulge H value could be tried in building joints and other areas where joints are not subjected to rolling traffic and snow plows. Laboratory work and field trials are needed to test the various possibilities.

It is often assumed that the best time to seal joints with a liquid sealant is during warm weather. If nothing else the warm weather should help with adhesion to the joint walls due to the

higher temperatures and most probably a lower moisture presence in the joint. In order to utilize both the compression and the tension cycles the joints would need to be sealed during "medium" temperatures (these are temperatures of the slab, rather than air temperatures). In some regions of the world such medium temperatures may be on the cool side and may need special precautions in joint sealing. Additional laboratory and field work accompanied by field installations should be helpful. However, joint seals which are placed even during relatively warm weather in Spring or Fall should survive better than those placed during hot Summer weather.

During the seal design examples above certain numerical values were used in the calculations. The slab length L has a fairly constant value in any project. On the other hand, slab movement for design purposes can be a rather uncertain value. One or more of the joints in any concrete slab system may not function properly and the joint may not change its width much. Therefore some knowledge of joint movement variability is needed. The slabs may develop different friction characteristics over different subbases. The temperature differences between Summer and Winter may vary from year to year. There may be other varied influences at work such as moisture expansion and drying shrinkage. When calculating the design joint movement of .24 inches fixed values for slab length, coefficient of expansion, and temperature difference were used. Also a fixed value for coefficient of variation is built into Figure 6. Since slab friction, moisture in the slabs and other smaller items were neglected, the value of .24 inches is slightly "soft". However, it is probably conservative.

Some data on joint expansion and contraction is already available. More gathering is needed and above all work should be started to set up reasonable limits and specifications for joint movements on which at least a sizable number of engineers and users can agree as practical and useful.

The maximum allowable tensile strain in the sealant was assumed as 20% in the calculations above. Is this reasonable if a full recovery (memory) of the sealant is expected after a yearly cycle in tension and compression? Is this the same for rubber asphalt and silicone sealants? The answer is: probably no, but the differences are not known. It could be expected that different materials will have different strain characteristics and different other properties. Urgent work is needed in this area.

The maximum allowable bulge was assumed to be .25 inches. This may or may not be the best value for every case but it could be a good starting point. This bulge may vary if the sealant is not fully strain recoverable.

The depth of the sealant (D) will vary from spot to spot in the joint. For design purposes the largest (maximum) depth is

42 Field Molded Joint Seals

the joint should be used because it is the most critical as far as induced strains are concerned. The width W_o at the time of sealing also may vary slightly. Here the lowest width value should be used as a design value. Field studies should be made on this subject. It must be pointed out that the benefits of the compression cycle can also be traded for deeper seals and narrower joints.

In all the above manipulations the basic assumption is that the tensile strain in the outer skin of the sealant is the critical area of concern. This makes various calculations simpler. It would be desirable to look at shear distribution in the sealant plug. Also the uniformity of tensile strain along the sealant surface should be studied.

After all these discussions it can be concluded that the design of pavement or any other live, constantly moving joint seal is not a simple matter. It also is apparent that the design values used are sometimes on the "soft" side and more laboratory and field work is needed to improve our engineering calculations. It is also apparent that installations of sealants in joints without at least simple design considerations will often lead to failures.

CONCLUSIONS

The discussion and calculations presented in this paper are basically an extension of work published earlier (1),(4). Initially only the joint expansion and tension case for the sealant in a joint was considered. In this paper both, joint expansion and contraction were covered which puts the sealant through tension and compression. The conclusions are based on certain simplified assumptions which are explained in the paper. The most important of these are: the sealant is poured or molded in the field as opposed to so called factory made compression seals; the sealant bulges out or curves down into the joint following a parabolic curve; tensile and compressive strains along the parabolic outer-most surfaces are assumed to be the critical strains; all compressive and tensile strains are assumed to be recoverable as the sealant goes through its yearly working cycle; at the time of sealing the sealant is assumed to have zero strain, compressive and tensile strain as the joint contracts and tensile strain as the joint expands. The main conclusions from this study are:

1. If the sealant must be placed flush with the slab top or outside surface and if there is any kind of traffic crossing the joint, the sealant should be placed in summer when the joint is at its minimum width. Such a sealant will be in tension through its service life with a relatively limited working range.

2. If the sealant must be placed flush with the slab surface but there is no traffic on it and the sealant is permitted to bulge out of the joint the sealant can be placed at intermediate temperatures (Spring or Fall). Such sealant will be subjected to both tension and compression and its actual working range can be increased by a factor of maybe 2 or more depending on the sealant shape factor and other considerations. This may be frequently applicable to building joints and other joints.
3. If the sealant can be placed slightly lower in the joint (recessed) so that the bulge during the compression cycle does not protrude out of the joint and the traffic cannot disturb it, both tension and compression capabilities of the sealant can be utilized. Again the working range of the sealant thus could be increased greatly and this method could be used in highway, building and other joints. Design calculations for tensile strains and the bulging of the sealant would have to be made for each job, but that is not difficult. In this case again the sealant would have to be placed in the joint during intermediate temperatures and the increase in working range will be considerable.
4. The procedures described in this paper are applicable to pavement, building and other joints and cracks with a relatively uniform width and rectangular cross-section.

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44 Field Molded Joint Seals

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APPENDIX 1

Maximum Tensile Strain Calculation Program in Basic.

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9  ***SMAXX*** CALCULATION OF MAXIMUM STRAIN IN LIQUID SEAL
10 W=1.5 this is Wx or the joint width at any time X
20 D=1 this is the original sealed depth
30 V=1 this is Wo or the original sealed width
35 X=W-V
37 A=((W*W) + (9*D*D*X*X)/(W*W)).5/2
39 B=(W.3)/(6*D*X)
41 C=((3*D*X)/W+(2*A))/W
43 L=A+B*(LOG (C))
50 S=((L-V)/V)*100
60 PRINT S used for y axes in Figs 2,3,4,5
70 END
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Month: J F M A M J J A S O N D J

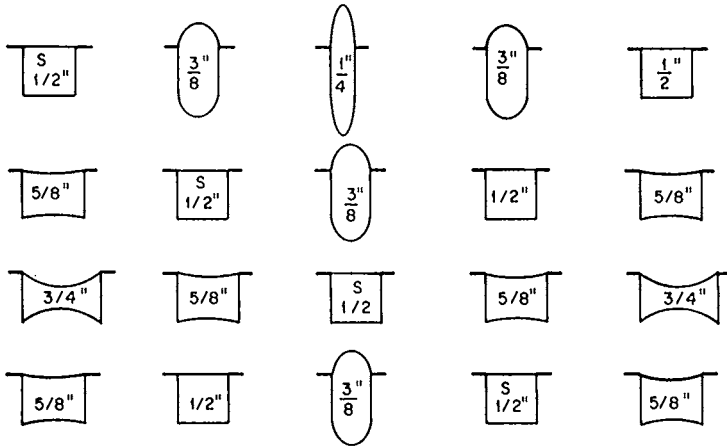


Fig. 1--Illustration of sealant cross section change during a yearly cycle as the function of sealing time (Winter, Spring, Summer or Fall) (S stands for sealing time)

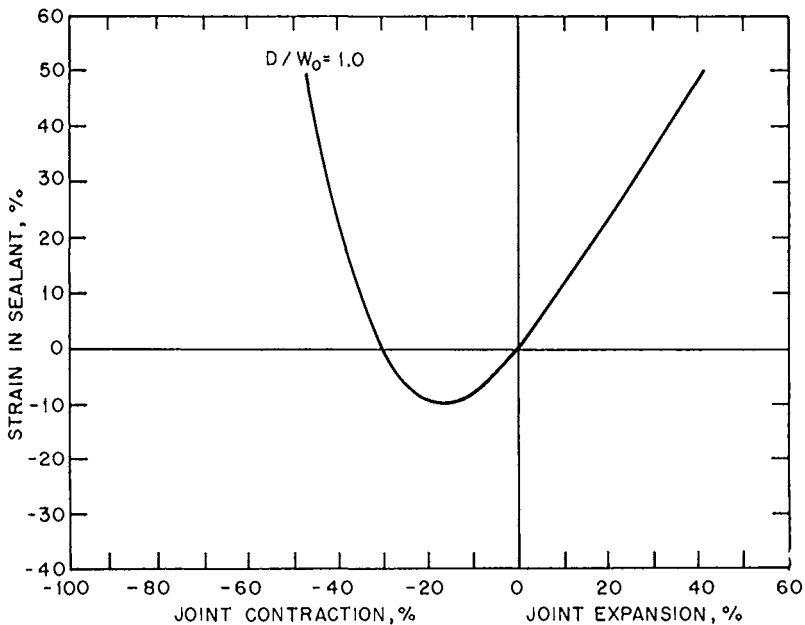


Fig. 2--Tensile and compressive strains for sealants with shape factor $D/W_o=1$ and $W_o=1$

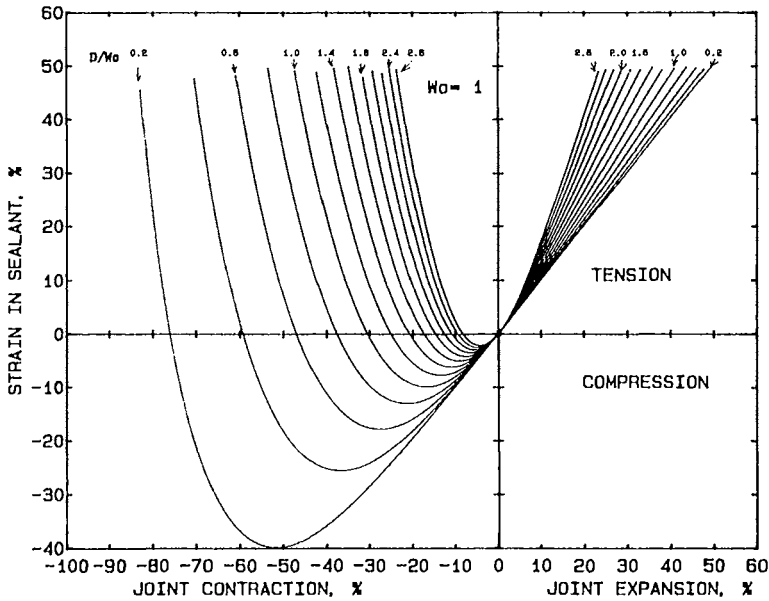


Fig. 3--Tensile and compressive strains for sealant with different shape factors (D/W_o , and $W_o=1$)

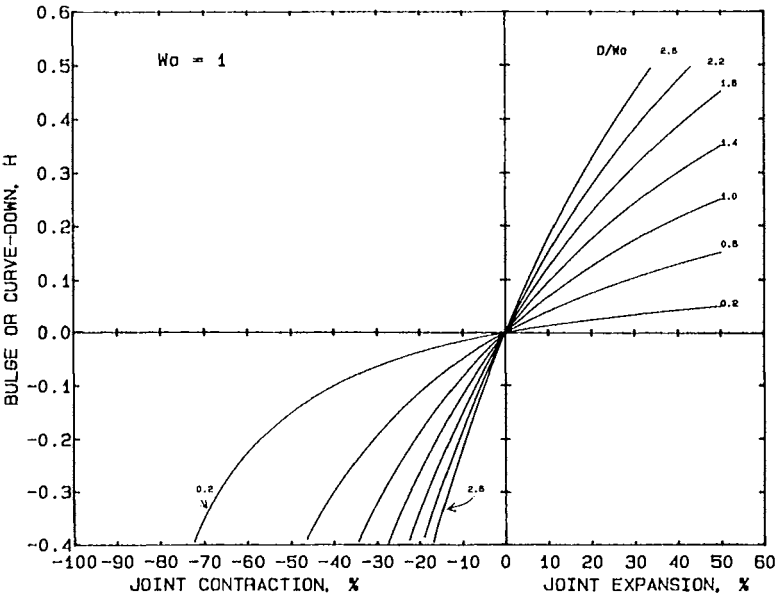


Fig. 4--Sealant bulging up and curving down value H for different shape factors (D/W_o , and $W_o=1$)

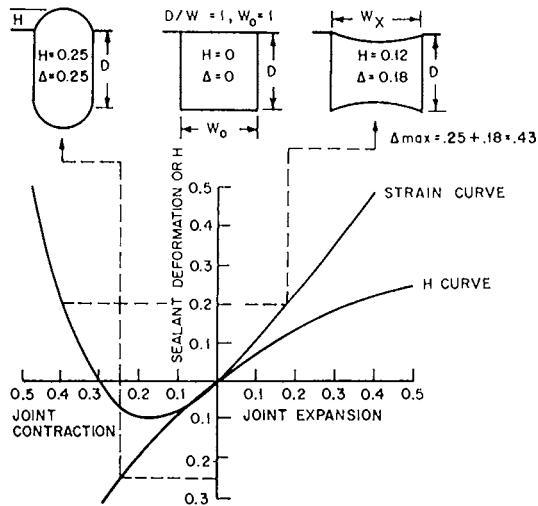


Fig. 5--Strain and bulge-depression curves for the case of $D/W_0=1$, $W_0=1$

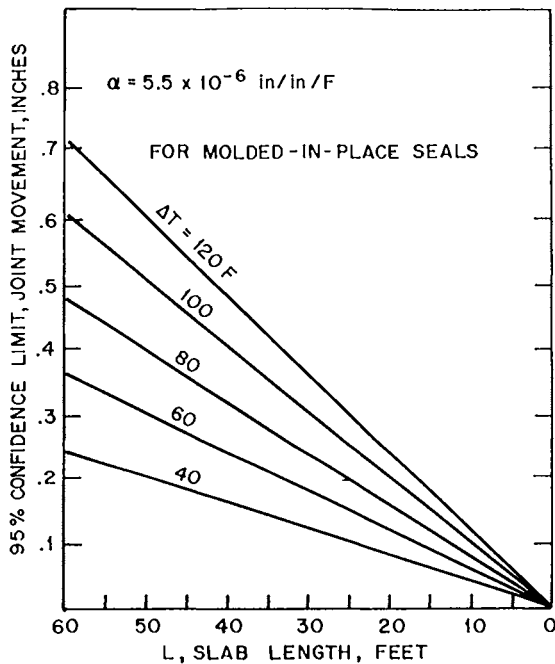


Fig. 6--Estimated design joint movement based on thermal coefficient, C.V.=30 percent, 95 percent confidence limit (Ref. 4)

Upgrading Seismically Vulnerable Bearings with Lead-Filled Elastomeric Bearings

by Ronald L. Mayes, Trevor E. Kelly, and Lindsay R. Jones

Synopsis: Failures in past earthquakes have indicated three major problem areas with existing bridges. The first is the vulnerability of existing rocker and roller type bearings and this is coupled with the second area, that of inadequate support lengths for superstructure girders. The third problem area is inadequate strength and ductility capacity of supporting substructures. By replacing vulnerable rocker and roller bearings with lead-filled elastomeric bearings, an effective and economical solution to all three deficiencies can be obtained by utilizing the base isolation and energy dissipating characteristics of the bearings.

The paper discusses the basic concept and philosophy of the base isolation design concept and outlines its application on a recently completed project--the Caltrans Sierra Point Overhead seismic retrofit project. This is an 8-span skewed bridge that has recently been retrofitted utilizing base isolation concepts. In its as-built condition, it was capable of resisting less than 25% of the Caltrans design ground motion for the site. Replacement of the existing pot bearings with lead-rubber bearings reduces the forces transmitted into the weak columns by a factor of 6, thus enabling the bridge columns to respond elastically to the design ground motion.

Keywords: base isolation; bridge bearings; earthquake resistant structures; energy dissipation; plastics, polymers, and resins; renovating

50 Lead-Filled Elastomeric Bearings

Ron Mayes, Trevor Kelly and Lindsay Jones have been involved with the development of the base isolation design concept for the past three years. They have worked as consultants on several projects that incorporate the concept and have been active in code development work. They are key employees of Dynamic Isolation Systems, Inc. of Berkeley, California.

INTRODUCTION

The issue of bridge rehabilitation has been receiving increasing political and engineering attention over the past five years and funding levels are beginning to approach levels where significant planning and construction progress can now be made. Major aspects of the bridge rehabilitation problem include the replacement or upgrading of deteriorated decks, an upgrade or realistic assessment of the load rating and the replacement of non-functioning or badly corroded bearings. There is an increasing trend towards the replacement of steel rocker and roller type bearings (shown in Fig. 1) with elastomeric bearings. By incorporating a lead plug and slightly increasing the height of elastomeric bearings, the level of earthquake resistance of a bridge can be increased by factors of 5 to 10. This issue forms the basis of the remainder of the paper.

The likelihood of an earthquake in different regions of the United States varies considerably. However, it must be noted that two of the largest U.S. earthquakes have occurred in the Eastern part of the United States. The U.S. Geological Survey has been studying these two earthquakes (Charleston and New Madrid) for the past ten years and an excerpt from their 1982 submittal to the Nuclear Regulatory Commission states:

"Because the geologic and tectonic features of the Charleston region are similar to those in other regions of the eastern seaboard, we conclude that although there is no recent or historical evidence that other regions have experienced strong earthquakes, the historical record is not, of itself, sufficient grounds for ruling out the occurrence in these other regions of strong seismic ground motions similar to those experienced near Charleston in 1886."

Every decade or so major conceptual advances are made in engineering. Such an example would be the use of prestressing in the concrete industry to enhance the economics of reinforced concrete structures. The base isolation design concept is another example of a major conceptual breakthrough in the field of earthquake engineering. The advantages offered by this concept for bridges include potential first-cost savings of up to 10% and

the elimination or significant reduction in earthquake damage. For bridge rehabilitation the use of lead-rubber elastomeric bearings can solve all three of the major earthquake problems and increase the earthquake resistance of an existing bridge by factors of 5 to 10. The concept has been extensively developed and applied in New Zealand with 37 bridges now designed. Of these, 26 have been designed with the lead-rubber bearing as shown in Table 1. The concept is receiving increased attention in the United States and two bridge projects have incorporated the concept and another is in the preliminary design phase. In addition to these bridge projects, four buildings have or will incorporate the concept.

THE SEISMIC REHABILITATION PROBLEM

The San Fernando earthquake of 1971 demonstrated the potential inadequacy of past design procedures in providing seismically resistant bridges. Since most existing bridges in service in the U.S. today were designed using pre-1971 design criteria, it follows that many of the nation's highway bridges in 37 seismically active states may have insufficient strength to resist seismic loading. Recognizing this problem, CalTrans developed an ongoing program of retrofitting bridges in California [6]. In addition, FHWA funded two major projects that were performed by Applied Technology Council. The first was the development of Seismic Design Guidelines for new bridges [1], and the second was the development of Seismic Retrofit Guidelines for existing bridges [2].

The CalTrans retrofit program primarily involves providing positive connections between superstructure segments and between the superstructure and supporting substructures. This solution aids in the lack of sufficient support lengths for bridge girders. At this time no attempt has been made by CalTrans to strengthen inadequate supporting substructures such as columns and piers primarily because of the expense.

Recent developments in New Zealand on the use of base isolation and energy dissipating mechanisms for the design of new bridges and buildings offer exciting possibilities for the retrofit of existing bridges. The concepts provide retrofit solutions for columns and piers with inadequate strength and for superstructure girders with inadequate support lengths. Superstructure displacements are significantly decreased during seismic excitation and the concepts provide a force-limiting mechanism for the supporting substructure.

The paper presents the application of the devices on two bridges. The first example is a typical three-span bridge and the impact on base isolation design of different seismic zones is examined. The second example is the CalTrans Sierra Point Overhead seismic retrofit project. This is an 8-span skewed bridge that has recently been retrofitted utilizing the base isolation concepts.

52 Lead-Filled Elastomeric Bearings

SEISMIC REHABILITATION CONSIDERATIONS

The Applied Technology Council has recently completed the development of seismic retrofit guidelines for highway bridges [2]. The guidelines are comprehensive and embody several new concepts which are significant departures from existing retrofitting practice. The guidelines include a preliminary screening procedure, methods for evaluating an existing bridge in detail, and potential retrofitting measures for the most common seismic deficiencies.

The three major problem areas identified in the ATC-6-2 report are:

1. The vulnerability of existing steel rocker and roller type bearings shown in Figure 1.
2. The inadequate strength and ductility of columns and piers.
3. The inadequate support lengths for girders.

Potential retrofit measures for these problem areas are presented in the ATC-6-2 report. Unfortunately, only a few have been utilized in practice, primarily because of the expense of some of the details that are required to alleviate all three problem areas and especially the inadequate strength and ductility of existing columns. Some of the measures presented in the ATC-6-2 report are summarized as follows:

- (a) Vulnerable Bearings: Replacement of bearings should be considered if their failure will result in collapse or loss of function of the superstructure. Types of bearings that have performed poorly in past earthquakes are shown in Figure 1 and suggested replacements include elastomeric bearings or more sophisticated energy dissipating devices.
- (b) Inadequate Strength and Ductility of Columns: Reinforced concrete columns or piers and the footings to which they are attached form a group of interacting components that are among the most vulnerable to earthquake damage. Some of the retrofit measures discussed are the use of force-limiting devices, increased transverse reinforcement for confinement of columns, reduced or increased longitudinal reinforcement, infill shear walls between column bents and strengthening of footings.
- (c) Inadequate Support Length for Girders: This is the area for which most of the current retrofit measures have been performed in the United States and primarily by CalTrans [6]. These measures have primarily included the provision of positive cable connections between the superstructure and supporting substructure. Measures used by CalTrans include longitudinal joint restrainers, transverse bearing restrainers and vertical motion restrainers. Other concepts presented