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ELASTIC PLATE SPALLATION
FINAL REPORT

by

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\[ \lambda \quad \text{Lamé constant} = \frac{\nu E}{(1 + \nu)(1 - 2\nu)} \]

\[ \mu \quad \text{Lamé constant} = \frac{E}{2(1 + \nu)} = \text{shear modulus} \]

\[ \nu \quad \text{Poisson's ratio} \]

\[ \pi \quad \text{Constant} = 3.14159 \]

\[ \sigma \quad \text{Stress} \]

\[ \tau \quad \text{Shear stress} \]
CHAPTER I

INTRODUCTION

Spallation is a failure condition caused by stress wave interaction from blast or impact loading of an object. The most common example is the conical hole produced in glass by the impact of a pellet. In more ductile materials, such as aluminum, a spall may be as subtle as the formation of an internal void or as dramatic as in glass, with the casting off of a chunk from the side of the plate opposite to the side impacted.

Previous work on stress waves and spallation has been done analytically, experimentally, and numerically. There is a great deal of work on infinite half-spaces, but only a few studies on plates, which are applicable and necessary in order to understand spallation of plates. Davids [1] has solved the problem of waves generated by a point load applied to a plate both before and after reflecting from the back face. A recent paper by Viswanathan and Biswas [2] studies waves in a plate produced by distributed loads. Their work is also analytical. Dally and Riley [3] have used dynamic photoelasticity to study stress waves in

---

¹Numbers in brackets, [], refer to the Bibliography.
a plate, but their work is not general. Ang and Newmark [4] studied stress waves by a finite difference technique but did not discuss reflections or spallation. Costantino [5] outlined the finite element method for studying stress waves but did not investigate the waves in detail or investigate reflections and spallation.

Figure 1 shows an experimentally produced spall in an aluminum plate. Obviously, large plastic and hydrodynamic forces were present in the total response of the plate to the impact. While a numerical scheme could be developed to approximate the total phenomenon, it is not necessary. The material must pass through the elastic range on the way to a spall condition. Investigation of the elastic waves alone will show which ones interact and where they interact to develop high stresses. These interactions and stresses may be interpreted for their contribution to spallation.

The research presented here uses dynamic finite elements to investigate elastic stress waves in a plate. The investigation will discuss all waves produced by point and distributed loads applied to a plate. Conclusions will be drawn as to their importance in spallation.
Figure 1. Experimentally Produced Spall in an Aluminum Plate
CHAPTER II

FINITE ELEMENT METHOD

A short description of the finite element method in general is given first to outline the salient points of the method. Then a more detailed description of the specific method for axisymmetric problems is given. The discussion of the axisymmetric dynamic finite element method also outlines the program used for this research. Detailed derivations of the items mentioned are in Appendix A. The program is listed in Appendix C.

General Method

The general finite element approach to the analysis of any continuous body may be listed in seven steps. These are as follows:

1. Divide the body into suitable pieces or elements;

2. Assume a displacement function for displacements in an element. Compatibility is insured because the functions include parameters defined in terms of nodal displacements;
3. Nodes are the intersections of the imaginary lines and planes used to divide the continuum into elements. The geometric locations of the nodes must be recorded and also which nodes correspond to each element;

4. Using force equilibrium at the nodes and virtual work, the elements are combined to a system of equations approximating the body. The equations involve stiffness, displacement, and applied forces;

5. If the problem is dynamic, the inertia must be accounted for in the equations developed in the previous step. For transient wave problems, the mass is lumped at the nodes. Standing wave problems should be solved with the "consistent" mass matrix which is derived by virtual work with the inertia forces treated as distributed body forces. The mass matrix times accelerations of the nodes is added to the static equations of step (4);

6. The system of equations is solved for nodal displacements;

7. Once the nodal displacements are known, they are used to find strains element by element which are in turn used to find stresses throughout the body.

In a static problem, the equations formulated in step (4) consist of stiffness terms times nodal displacements.
set equal to forces externally applied to the nodes. The nodal displacements are determined by an elimination technique or by inversion of the stiffness matrix. In a dynamic problem, the mass times acceleration terms create second-order, linear differential equations. These may be solved by integration, which may be prohibitively difficult, or by rewriting the acceleration in finite difference notation. The resulting algebraic equations are solved stepwise in time. The displacements, strains and stresses change with each time step.

The lumped mass approach is based on a paper by Costantino [5]. The following axisymmetric application is based mainly on a discussion by Zienkiewicz [6].

**Dynamic Finite Element Method for Transient Waves in an Axisymmetric Plate**

**Element Formulation**

A circular, symmetric, distributed load applied to an infinite plate produces an axisymmetric response in the plate. The infinite plate is approximated by an axisymmetric disk of finite radius where the radius is great enough so that reflections from the radial boundary do not interfere with reflections from the back face of the plate until after the time of interest. This axisymmetric model is easily divided into axisymmetric finite elements as shown in Figure 2. The elements are triangular in the R-Z plane for simplicity of formulation and are the same as plane
Figure 2. Cutaway of Axisymmetric Finite Elements and Plate
strain elements with the exception that thickness of an element is replaced by centroidal circumference. Also, they must be capable of circumferential strain and stress. Nodal points of plane theory become nodal rings in axisymmetric application. Circumferential effects are a function of radial effects only, so that only radial and axial displacements are required to fully describe the problem. Suitable displacement functions involve one constant for each degree of freedom of the element. This is a Rayleigh-Ritz approximation.

\[
\begin{align*}
UR &= \alpha_1 + \alpha_2 R + \alpha_3 Z \\
UZ &= \alpha_4 + \alpha_5 R + \alpha_6 Z
\end{align*}
\]

where \(\alpha_1\) through \(\alpha_6\) are the unknown constants.

R and Z are the coordinate position of the point whose displacement is desired.

UR = radial displacement of the point.

UZ = axial displacement of the point.

The constants, \(\alpha_i\), \((i = 1, 2, \ldots, 6)\), may be defined in terms of nodal displacements as is shown in Appendix A. The displacement of any point in an element is then defined by the nodal displacements and the geometric position of the point.
Strain Displacement Relations

From the definition of strain (Fung [7]),

\[ \varepsilon_{ij} = \frac{1}{2}(U_i, J + U_j, i) \]

and the assumption of axial symmetry, only four strains remain.

\[
\begin{align*}
\varepsilon_R &= \frac{\partial U_R}{\partial R} \\
\varepsilon_\theta &= \frac{U_R}{R} \\
\varepsilon_Z &= \frac{\partial U_Z}{\partial Z} \\
\gamma_{RZ} &= \frac{\partial U_R}{\partial Z} + \frac{\partial U_Z}{\partial R}
\end{align*}
\]

Equation set (2) can be arranged in matrix form (see Appendix A). These are constant strain elements.

\[
\{\varepsilon\}_e = [B]\{\delta\}_e \quad (3)
\]

where \([B]\) is a matrix of coefficients based solely on geometry of the element. The matrix \{\delta\}_e is the nodal displacements of the element.

\[
\{\delta\}_e = \begin{bmatrix}
UR_i \\
UZ_i \\
UR_j \\
UZ_j \\
UR_m \\
UZ_m
\end{bmatrix}
\]
Stress Strain Relation

Hooke's law for axisymmetric problems reduces to four stresses.

\[
[D] = \frac{E}{(1 + \nu)(1 - 2\nu)} \begin{bmatrix}
(l-\nu) & \nu & 0 \\
\nu & (l-\nu) & 0 \\
\nu & \nu & (l-\nu) \\
0 & 0 & 0 & \frac{(1-2\nu)}{2}
\end{bmatrix}
\]

\[
\begin{bmatrix}
\sigma_R \\
\sigma_\theta \\
\sigma_Z \\
\tau_{RZ}
\end{bmatrix} = [D] \begin{bmatrix}
\varepsilon_R \\
\varepsilon_\theta \\
\varepsilon_Z \\
\gamma_{RZ}
\end{bmatrix}
\]

Stiffness Matrix

Element stiffness may be calculated by the principle of virtual work. The matrix \( \{S\}_e \) is the nodal forces resisting deformation of an element. Then from the principle of virtual work:

\[
\{S\}_e \{\delta\}_e^T = \int_V \sigma \varepsilon dV.
\]

Substituting equations (3) and (4) and integrating,

\[
\{S\}_e \{\delta\}_e^T = 2\pi \cdot \text{RBAR} \cdot A \cdot \{\delta\}_e^T [B]^T [D] [B] \{\delta\}_e
\]

\[
\{S\}_e = 2\pi \cdot \text{RBAR} \cdot A \cdot [B]^T [D] [B] \{\delta\}_e.
\]

RBAR is the radial distance to the centroid of the element. Therefore the stiffness matrix is

\[
[K]_e = 2\pi \cdot \text{RBAR} \cdot A \cdot [B]^T [D] [B].
\]
The stiffness for the system of elements representing the continuous body may be obtained by adding the element stiffnesses in the proper way. This may be done in a tedious fashion by writing force equilibrium at each node of the entire system and also element by element. The terms of the element equilibrium equations are added into the appropriate positions in the set of equations for the entire system. This may be simplified by "globally" numbering the nodes in any orderly sequence and then listing their displacements in a \( \delta \) array for the entire system sequentially from smallest node number to largest. Each element's nodes are called \( i, j, \) and \( m \), numbering counterclockwise (See Figure 3). Nodes \( i, j, \) and \( m \) have values prescribed by the global numbering. Then any number, \( k_{12} \), for example, in the element stiffness matrix adds into a position in the assembled stiffness matrix calculable from the node numbers \( i, j, \) or \( m \) which correspond to \( k_{12} \).

The element stiffness matrix is \( 6 \times 6 \) because the element has six degrees of freedom. The order of the assembled stiffness matrix is \( 2n \times 2n \) where \( n \) is the number of nodes and \( 2 \) is the number of degrees of freedom per node. However, the assembled stiffness matrix is banded and sparsely populated for the element system of this research. In fact, the maximum number of non-zero terms on any row is fourteen. This is because the greatest number of nodes adjacent to any one node is six; so one node plus six adjacent times two degrees of freedom is fourteen.
Figure 3. Typical Node and Element Numbering Scheme
Great savings in core storage are achieved by making use of this fact. An additional matrix called NADJ, composed of the node numbers of each node and the nodes adjacent to it is used to condense the assembled stiffness matrix, KASY, from $2n$ by $2n$ to $2n$ by $14$. A further explanation is given in Appendix A.

Mass Matrix

The lumped mass matrix is used in this research because of the assumption made by Costantino [5] that the consistent mass matrix would not allow wave propagation and in fact give rise to an infinite wave speed. That is, all points in the body would feel the disturbance at the same time. Appendix B shows that Costantino's assumption is valid and compares the consistent mass matrix and lumped mass matrix in a simple closed form problem. The consistent mass matrix is derived in Appendix B.

The lumped mass matrix has proven to give appropriate results. The wave speeds are correct and stress distributions follow the correct shape and approximate the correct magnitude as is discussed in Chapter III.

The lumped mass matrix does not require any derivation. The mass of an element is equally divided among its three nodes. A node shared by six elements then has the mass of two elements concentrated at it. A node shared by three elements has the mass of one element and so forth. The lumped mass matrix is diagonal. Formulation of the
mass matrix for this axisymmetric problem is shown in Appendix A.

As an example of the lumped mass matrix consider the mass times acceleration for the following one dimensional bar example.

5 nodes, 4 elements
m_0 = mass of one element

\[
\begin{bmatrix}
\frac{1}{2} & 0 & 0 & 0 & 0 \\
0 & 1 & 0 & 0 & 0 \\
0 & 0 & 1 & 0 & 0 \\
0 & 0 & 0 & 1 & 0 \\
0 & 0 & 0 & 0 & \frac{1}{2}
\end{bmatrix}
\begin{bmatrix}
u_1 \\
u_2 \\
u_3 \\
u_4 \\
u_5
\end{bmatrix}
\]

Equations of Motion
Consider a simple spring and mass system.
Statically; \[ \sum F = 0 \]
\[-k\delta + W = 0 \]
\[ k\delta = W \]

Dynamically;
\[ \sum F = ma \]
\[-kx + W = mx \]
\[ mx + kx = W \]

This approach holds for a complicated system of masses and springs which is exactly the approximation of dynamic finite elements with mass lumped at the nodes.

In matrix form,
\[ [M]\ddot{\delta} + [K]\dot{\delta} = \{R\} \tag{6} \]

where
\[
\{\dot{\delta}\} = \begin{bmatrix}
UR_1 \\
UZ_1 \\
UR_2 \\
UZ_2 \\
\vdots \\
UR_n \\
UZ_n
\end{bmatrix}
\]

n is the number of nodes, and the matrix \{\ddot{\delta}\} is a columnar array of nodal acceleration arranged in the same order as \{\delta\}. 
To solve this set of equations, the accelerations are rewritten in finite difference notation. Thus,

\[
\ddot{U} \bigg|_{t>0} = \frac{U(t + \Delta t) - 2U(t) + U(t - \Delta t)}{\Delta t^2}
\]

Substituting the above relation into the equations of motion (6), each equation may be solved algebraically for \( U(t + \Delta t) \). The equations are elastically coupled, but this does not interfere with the solution which is done node by node stepwise in time by increments of \( \Delta t \). The time step \( \Delta t \) can be no larger than the time required for the dilatation wave to cross one element. The elements are right isosceles triangles so \( \Delta t_{\text{max}} = 0.707 \frac{DE}{C_1} \), where \( DE \) is the length of one leg of the triangle and \( C_1 \) is the dilatation wave speed. For convenience \( \Delta t = 0.5 \frac{DE}{C_1} \) was used. This time step produces results indistinguishable from \( \Delta t_{\text{max}} \).

The initial time step is a special case in which the condition

\[
\ddot{U} \bigg|_{t=0} = \frac{U_t + \Delta t - U_t - \Delta t}{2\Delta t} = 0
\]

must be used.
CHAPTER III

ACCURACY OF PROGRAM

Ang and Newmark [4] solved a problem involving the blast loading of earth with a finite difference technique. Costantino [51] used the same problem to check his finite element program which used rectangular elements. The program used in this research (Appendix C) used triangular elements (Figure 3) and produced similar results to the previous two solutions. Figure 4 shows the load distribution on the surface. The similarity of the curves in Figure 5 shows each method is satisfactory.

Results from the program in Appendix C were utilized to make Figure 6. The solid lines are theoretical wave positions calculated from the well-known formulas for wave speeds in a solid (see Mason [8]).

\[ V_D = \sqrt{\frac{\lambda + 2\mu}{\rho}} \quad \text{Dilatation wave speed} \quad (9) \]
\[ V_S = \sqrt{\frac{\mu}{\rho}} \quad \text{Shear wave speed} \quad (10) \]

where \( \lambda \) and \( \mu \) are the Lamé constants.

Although the velocities computed are delayed in time, they are correct. The dilatation wave speed was located by the peak compressive axial stress generated by
Figure 5. Axial Stress on Centerline of Earth Model

- Finite element by Medaglia
- Finite element by Costantino
- Finite difference by Ang and Newmark

\[ E = 4 \times 10^6 \text{ PSI} \]
\[ v = 0.26 \]
\[ \gamma = 125 \text{ pcf} \]
Figure 6. Position of Wave Peaks Produced by Full Sine Pulse Applied as a Point Load
a sinusoidal point load, and the shear wave position was located by the peak radial displacements generated by the same sinusoidal point load. The delay in time is caused by the inertia of the lumped masses resisting the motion imparted by the forcing function. A sample print-out is included in Appendix C.

Figure 7 shows the temporal history of stress at a certain depth on the centerline. The theoretical solution is due to Davids [1]. An exponentially decaying step point load (Figure 8) was applied on the centerline, normal to the surface of the plate. The severe oscillations are due to the element size and the discontinuous nature of the forcing function. The discontinuity was reduced by applying a ramp-step load shown in Figure 9. Davids stated that for a step load the stress at a point would approach a value greater than zero for large times as shown in Figure 10. Figure 10 shows that the longer the ramp time, the smaller the oscillations. In fact it can be seen from Figure 5, that the oscillations become unimportant when the discontinuities in the forcing function are small. When a full sine forcing function is applied, the oscillations are unnoticeable as can be seen in Figure 11. The tensile peak at 1 micro-second is due to the shear wave.

A more precise understanding of the nature of the oscillations may be achieved by writing the finite element equations for a one dimensional bar and solving them by Laplace transforms. This procedure is carried out in detail
Figure 7. Stress Response at a Point on the Centerline due to an Exponentially Decaying Point Load

\[ F(t) = F_0 e^{-Bt} \]

\[ B = 2.435 \times 10^5 \text{ sec}^{-1} \]
Figure 9. Ramp Step Loads

Figure 10. Response to Ramp Step Loads
\[ F(t) = F_o \left[1 + \sin\left(\frac{2\pi t}{TTF} - \frac{\pi}{2}\right)\right] \]

a. Full Sine Pulse

\begin{align*}
Z &= .025" \\
H &= .23" \\
E &= 10.7 \times 10^6 \text{ psi} \\
\nu &= .3125 \\
\gamma &= .101 \text{ pci} \\
DE &= .01 \text{ in} \\
F_o &= 100# \\
TTF &= 0.95 \mu\text{sec}
\end{align*}

b. Response to Full Sine Pulse Point Load

Figure 11. Full Sine Pulse and Response
in Appendix B. The result is that at time zero for a bar approximated by two elements, using the lumped mass approach the reaction at one end is equal to the applied load at the other end. The same is true when the consistent mass matrix is used. This is because two elements are just one finite difference space for acceleration. For four lumped mass elements, the reaction is one-fifth the applied load. Using four consistent mass elements the reaction equals the applied load at time zero. The exact solution is that the reaction at time zero is zero. The more lumped mass elements that are used, the smaller the reaction. The consistent mass elements always give a reaction equal to the load, no matter how many elements are used, which indicates that the disturbance is propagated at an infinite speed.

The reason for the non-zero (albeit small) reaction in the finite element approximation is that the solution is a Fourier series with a finite number of terms. A plot of such a series is a curve which oscillates about the exact answer. With enough terms in the series, the oscillations become indistinguishable, except at discontinuities. At a discontinuity, a jump known as Gibb's phenomenon always occurs and is proportionate to the size of the discontinuity. The number of finite elements along the length is analogous to the number of terms in a Fourier series for approximation of waves which propagate axially. Therefore, the smaller the element size, the more terms in the series
and the better the approximation. Further, the first term in the series is the mean value of the series, hence the size of the reaction at time zero is just a measure of accuracy.

As a final check of the program, a step load was applied to a long cylindrical bar. Figure 12 shows the results for two element sizes at about the same instant in time. The smaller elements give a more accurate answer in terms of percent error and rapidity of convergence behind the wave front. The small difference in the two results shows the larger element size is actually a good approximation, and the results are influenced strongly by Gibb's phenomenon. The number of nodes in the two problems is equal. The only difference in the two bars is the element size. The grids are identical, each having five nodes radially and 168 nodes axially. The different element size changes the dimensions of the bar, but the number of terms in the approximation is the same. Therefore, the number of terms in the Fourier series for each element size is the same and the accuracy of each should be similar. The series for the smaller elements is somewhat more accurate because the coefficients of the terms in the series are better proportioned.

It is clear from the previous assessment of accuracy that the lumped mass matrix does give satisfactory results with sufficiently small elements and continuous forcing functions. The accuracy improves with finer grids (more
Figure 12. Long Bar Approximated by Two Element Sizes
computer space). While the accuracy of the stresses was limited by the capacity of the computer used, they do compare favorably with other numerical and exact solutions, and the necessary waves are generated at the proper speeds and locations to allow study of spallation. Limitations do exist, however, on the allowable sharpness of a discontinuous input load.
CHAPTER IV

RESULTS

Although all four stresses (radial, circumferential, axial, and shear) are present, examination of the stress distribution has shown that the axial stress is dominant, especially in the area where a fracture may develop. Experimentally produced spalls are usually tensile failures. Since the axial stress is the most prominent, and undergoes the most dramatic fluctuations, and the applied stress is in the axial direction, the results shown are for axial stress. This will serve to illustrate the major response of the plate. It would be futile to attempt to convey all or most of the information generated by the program in the form of graphs. Each problem discussed produced approximately forty stress distributions and nodal displacement displays which were each for separate instant in the response history of the plate. Condensation of these results to comments and a few graphs became essential. Direction of load application was always normal to the front face of the plate.
Point Load

The results of the point load application compare favorably with Davids [1] predictions. Although the step load response oscillates severely for reasons explained in Chapter III and Appendix B, Figure 6 shows the response does follow the proper values in time and space. The response is smooth when a full sine wave is applied as the forcing function (Figure 11).

The full sine forcing function has a shape which produces a clear peak in the response which can be easily followed as the waves propagate. The lack of oscillation and the obvious peak made the full sine forcing function the convenient choice for investigation of the stress waves. For the point load problem, a pulse of 100 pounds maximum amplitude was applied to the centerline in the axial direction. Figure 13 shows the axial stress distribution for several times, and the significant features are labeled.

The stress on the centerline is the only one shown because the magnitude is highest on the centerline. Examination of the semi-graphic printout (see sample printout in Appendix C) showed the stress waves moving out symmetrically from the point of load application in a pattern similar to the ripples produced by a stone dropped in a puddle. Figure 14 shows this pattern in idealized form before and after reflection of the dilatation wave from the back face. Figure 13 shows the response of the plate
3.85 μsec
Shear Wave Peak (S₃)

4.68 μsec
Dilatation Wave Peak (S₁)

a. Before Reflection of Dilatation Wave

TTF = 3.68 μsec
E = 10.7 x 10⁶ psi
v = .3125
γ = .101 psi
DE = 0.04 in
F₀ = 100#

Constructive Super Position of (S₃) and (S₁)

6.19 μsec

7.19 μsec

b. After Reflection

Figure 13. Axial Stress on Centerline Caused by Point Load, TTF = 3.68
subjected to a full sine pulse whose period is about the same as the time required for the dilatation wave to cross the thickness of the plate. Figure 15 shows what happens when the period of the pulse is equal to the time required for the dilatation wave to cross five-eighths of the thickness of the plate.

Distributed Load Over a Small Radius

A small radius is one which is equal to or less than about one-third of the plate thickness. The wave initially has a plane front, but seems to round off to a shape like that of the point load and produces a similar response.

The full sine pulse was used for distributed loads, also. A maximum amplitude of 1,000 psi was chosen for convenience.

The wave generated from the front face propagates in the axial direction for radii less than the radius of loading. From the point at the outer edge of the distributed load, the wave moves out like the waves produced by a point load. Since the dilatation wave moves away perpendicularly from the front face, it should not be followed by a shear wave. On the centerline it is not immediately followed by a shear wave.

But at the extreme radius of loading the dilatation wave is followed by a shear wave immediately. It is produced because the dilatation wave has a radial component at that point. As time goes on, the shear wave spreads from the
Figure 14. Idealized Wave Patterns Produced by Point Load

TTF = 2.426 μsec
E = 10.7 x 10^6
ν = .3125
γ = .101 pci
DE = 0.04 in.
F₀ = 100#

Figure 15. Axial Stress on Centerline Caused by Point Load

TTF = 2.426
radius of loading radially and axially. The radial propagation is actually a radial dilatation wave caused by the Poisson effect. The reflection of this wave from the front face generates the shear wave. With the small radius of loading, the shear wave spreads to the centerline and follows the axial dilatation wave. The smaller the radius of loading, the sooner the shear wave reaches the centerline. In addition, the part of the plane wave at the radius of loading attenuates causing the plane wave to apparently "round-off" to a shape similar to the shape caused by a point load. In comparison to a point load, the shear wave initiation on the centerline is delayed when the load is distributed. Figures 16, 17, 18 and 19 show axial stresses on the centerline for four different radii of loading at different times. Again the centerline is the location of the maximum stress now because of the rounding off of the dilatation wave.

Figure 20 compares the peak axial stress on the centerline as a function of depth for four radii of loading and a point load. The point load decays by the inverse of depth squared. The distributed loads approach a non-decaying plane wave as the radius of loading increases. Figure 21 shows the idealized wave patterns at three times.

**Distributed Load Over an Intermediate Radius**

An intermediate radius is less than or equal to the plate thickness and greater than one-third of the plate
T = 3.85 μsec

a. Just After End of Loading Pulse

T = 4.182 μsec

b. Shear Wave Fully Developed on S

T = 6.86 μsec

c. Maximum Superposition of S₃ + S₁

TTF = 3.68 sec
E = 10.7 x 10⁶ psi
v = 0.3125
γ = 0.101 psi
DE = 0.04 in.
P = 2,160 psi
a = 0.08 in.

Figure 16. Axial Stress on Centerline Caused by Load over Small Radius, a = 0.08 in.
Figure 17. Axial Stress on Centerline Caused by Load Over Small Radius, a = .16 in.

- **T = 3.85 μsec**
  - Just after End of Loading Pulse

- **T = 4.68 μsec**
  - Shear Wave Fully Developed on \( S_3 \)

- **T = 6.86 μsec**
  - Maximum Superposition of \( S_3 + S_1 \)

- **TTF = 3.68 μsec**
- **E = 10.7 \times 10^6 \text{ psi}**
- **\( v = .3125 \)**
- **\( \gamma = .101 \text{ pci} \**

- **DE = 0.04**
- **\( F_0 = 1,050 \text{ psi} \)**
- **\( a = 0.16 \text{ in.} \)**
a. Just after End of Loading Pulse

b. Shear Wave Fully Developed on CL

c. Maximum Superposition of $S_3 + S'_1$

\[ T = 3.85 \mu \text{sec} \]

\[ T = 5.353 \mu \text{sec} \]

\[ T = 6.86 \mu \text{sec} \]

$TTF = 3.68 \mu \text{sec}$

$DE = 0.04 \text{ in.}$

$E = 10.7 \times 10^6 \text{ psi}$

$F = 1,000 \text{ psi}$

$\nu = 0.3125$

$\sigma = .24 \text{ in.}$

$\gamma = .101 \text{ pci}$

Figure 18. Axial Stress on Centerline Caused by Load Over Small Radius, $a = .24 \text{ in.}$
a. Just after End of Loading Pulse

$T = 6.022 \mu s$ 

b. Shear Wave Fully Developed on $\xi$

$T = 7.19 \mu s$

$TTF = 3.68 \mu s$ 

$E = 10.7 \times 10^6$ psi

$\nu = 0.3125$

$\gamma = 0.101$ psi

$\alpha = 0.32$ in.

$F = 1,000$ psi

$DE = 0.04$

Figure 19. Axial Stress on Centerline Caused by Load Over Small Radius, $a = 0.32$ in.
Figure 20. Peak Compressive Axial Stress on Centerline versus Depth

Figure 21. Idealized Wave Patterns Produced by Load Over Small Radius

S₂ is an "edge" wave (Ref [2])
thickness. The maximum axial stress develops on the centerline, but high axial stresses also develop off the centerline.

The high axial stress off the centerline becomes more pronounced as the radius of loading increases. If the axial stress distribution is thought of as a contoured surface, a saddle of lower axial stress exists between the on and off centerline regions of high axial stress.

The on-centerline axial tensile stress develops first. The larger the radius of loading, the closer its location of development approaches a depth of about 70 percent of the plate thickness. The larger the radius of loading, the deeper the wave on the centerline travels at its applied stress value, resulting in less attenuation and higher peak tensile axial stresses for larger loading radii. The location of this stress buildup corresponds well with the location of the superposition of the edge and dilatation waves. This location is dependent on the wave speeds which are functions of the material properties.

The off-centerline tensile stress increases to a maximum located at a radius less than the loading radius but at a time well after the time the high tensile stress develops on the centerline. At the time the off centerline stress is maximum, it is the greatest tensile stress in the plate, but lower than the previously developed highest stress on the centerline. The location of the off-centerline maximum was observed to occur at half the
loading radius for a loading radius equal to the plate thickness.

Figure 22 shows idealized wave patterns for this case. Figures 23 and 24 show axial stress distribution. Notice that the curves are for radii off the centerline as well as on it. The applied loads were uniform distributed loads.

**Distributed Load over a Large Radius**

When the loading radius is greater than the plate thickness, the off-centerline tensile axial stress develops first, propagates toward the centerline and exceeds the stress on the centerline. Once it reaches the centerline it continues to increase briefly, then attenuates. It stays on the centerline and propagates toward the front face. While developing and moving toward the centerline, the stress also moves somewhat toward the front face. After it reaches the centerline another "hot-spot" develops off the centerline with lower stresses between it and the centerline. The new hot-spot develops at the same depth as the current maximum centerline stress, moves with it toward the front face and also toward the centerline. This new, late developing hot-spot is smaller than the maximum centerline stress. It is the same as the off centerline hot-spot developed by a load applied over an intermediate radius. Figure 25 shows idealized wave patterns for this case.
Figure 22. Idealized Wave Patterns for Loading over Intermediate Radius

a) \( R = 0.46 \) in.
   \( T = 6.52 \) \( \mu \)sec

\[ S_1 + S_2 \]

b) \( R = 0.0 \) in.
   \( T = 6.52 \) \( \mu \)sec
   \( TTF = 3.63 \) \( \mu \)sec
   \( E = 10.7 \times 10^6 \) psi
   \( v = 0.3125 \)
   \( \gamma = 1.01 \) pci
   \( F_o = 1,000 \) psi
   \( D_e = 0.04 \)
   \( a = 0.92 \) in.

\[ S_1 + S_2 \]

Figure 23. Axial Stresses Due to Load Over Intermediate Radius, \( T = 6.52 \) \( \mu \)sec
a) \( R = 0.46 \text{ in.} \)
\( T = 7.53 \mu \text{sec} \)

b) \( R = 0.0 \text{ in.} \)
\( T = 7.53 \mu \text{sec} \)

Figure 24. Axial Stresses due to Load Over Intermediate Radius, \( T = 7.53 \mu \text{sec} \)
Figure 25. Idealized Wave Patterns for Loading over Large Radius
Figures 26 and 27 show axial stress at different radii and times.

Table 1 shows important stress values, times and locations for each loading condition. Comparison of the times in the last two columns shows whether the maximum stress is due to superposition of the reflected dilatation ($S_1^1$) with the incident shear ($S_3$) or with the edge wave ($S_2$). The table summarizes several figures in this chapter.
Figure 26. Axial Stress Caused by Load Over Large Radius, $T = 4.43$ and $4.60$ μsec
\[ R = 0.66 \]
\[ T = 5.27 \ \mu\text{sec} \]

\[ R = 0.38 \]
\[ T = 5.27 \ \mu\text{sec} \]

\[ R = 0.0 \]
\[ T = 5.27 \ \mu\text{sec} \]

\[ 1210 \ \text{psi} \]

\[ \text{Figure 27. Axial Stress Due to Loading Over Large Radius, } T = 5.27 \ \mu\text{sec} \]
<table>
<thead>
<tr>
<th>$F_0$</th>
<th>Load Radius</th>
<th>TTF μsec</th>
<th>Plate Thickness</th>
<th>Axial Stress</th>
<th>Location $R$</th>
<th>Location $Z$</th>
<th>Time μsec</th>
<th>Theoretical Time for $S_1 + S_3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>100#</td>
<td>0</td>
<td>3.68</td>
<td>0.92</td>
<td>220 psi</td>
<td>0.0</td>
<td>0.58</td>
<td>7.19</td>
<td>6.9</td>
</tr>
<tr>
<td>100#</td>
<td>0</td>
<td>2.43</td>
<td>0.92</td>
<td>260 psi</td>
<td>0.0</td>
<td>0.58</td>
<td>6.11</td>
<td>6.3</td>
</tr>
<tr>
<td>2,160 psi</td>
<td>0.08</td>
<td>3.68</td>
<td>0.92</td>
<td>130 psi</td>
<td>0.0</td>
<td>0.54</td>
<td>6.86</td>
<td>6.9</td>
</tr>
<tr>
<td>1,000 psi</td>
<td>0.16</td>
<td>3.68</td>
<td>0.92</td>
<td>190 psi</td>
<td>0.0</td>
<td>0.54</td>
<td>6.86</td>
<td>6.9</td>
</tr>
<tr>
<td>1,000 psi</td>
<td>0.24</td>
<td>3.68</td>
<td>0.92</td>
<td>360 psi</td>
<td>0.0</td>
<td>0.50</td>
<td>6.86</td>
<td>6.9</td>
</tr>
<tr>
<td>1,000 psi</td>
<td>0.32</td>
<td>3.68</td>
<td>0.92</td>
<td>550 psi</td>
<td>0.0</td>
<td>0.54</td>
<td>7.19</td>
<td>6.9</td>
</tr>
<tr>
<td>1,000 psi</td>
<td>0.92</td>
<td>3.68</td>
<td>0.92</td>
<td>1,290 psi</td>
<td>0.0</td>
<td>0.70</td>
<td>6.52</td>
<td>6.9</td>
</tr>
<tr>
<td>1,000 psi</td>
<td>0.92</td>
<td>3.68</td>
<td>0.92</td>
<td>820 psi</td>
<td>0.46</td>
<td>0.42</td>
<td>7.53</td>
<td>6.9</td>
</tr>
<tr>
<td>1,000 psi</td>
<td>0.92</td>
<td>2.76</td>
<td>0.68</td>
<td>980 psi</td>
<td>0.38</td>
<td>0.54</td>
<td>4.60</td>
<td>5.01</td>
</tr>
<tr>
<td>1,000 psi</td>
<td>0.92</td>
<td>2.76</td>
<td>0.68</td>
<td>1,210 psi</td>
<td>0.0</td>
<td>0.42</td>
<td>5.27</td>
<td>5.7</td>
</tr>
</tbody>
</table>
CHAPTER V

SUMMARY AND CONCLUSIONS

Detailed, quantitative failure criterion are not discussed in this presentation, but a qualitative discussion is given on the nature of the response of an elastic plate to various impulsive loading conditions.

Point Load

The axial point load on the centerline sends a dilatation wave in all directions, but primarily (in terms of magnitude) in the axial direction. The Poisson effect causes the radial propagation which reflects from the front face as the wave expands. This reflection produces the shear wave, which follows the axial dilatation wave at the shear wave speed. The shear wave has particle motion transverse to the direction of propagation. The dilatation wave has particle motion in the same direction as propagation.

The shear wave produces an axial stress opposite in sign from the axial stress caused by the dilatation wave. When the speedier dilatation wave reflects from the back face (a free surface), the reflection returns at the dilatation wave speed with axial stress of the opposite
sign. The reflection also produces a shear wave, but this shear wave occurs too late to influence spallation. The reflected dilatation wave and incident shear wave, therefore, have axial stress of the same sign and superpose constructively to cause a high tensile stress on the centerline. This high stress can be greater than the tensile strength of the material and lead to a spall. Although the front face may be damaged by the impact or blast, this damage does not necessarily pierce the plate and, until the shear wave and reflected dilatation wave superpose, stresses are not great enough to fracture the material. For a point load, the crack initiates on the centerline and spreads radially. The location on the centerline may be found from graphs similar to Figure 6 for step loads and Dirac delta loading. However, the sine pulse attenuates in a way such that the tensile stress develops at a shallower depth than predicted by Figure 6.

Small, Uniform Loading Radius

Initially, the stress waves differ from the point load, but after the shear wave develops on the centerline, the response is very similar to that of a point loaded plate. The dilatation wave has decayed away from the centerline so it seems to round off, but the front is a straight line radially. This attenuation also occurs on the centerline (after a certain depth) but to a lesser degree. The stress will build up in the same way as for the point
load. The fracture condition develops on the centerline and spreads radially. Attenuation of the sine pulse results in the location of the tensile stress to be at a shallower depth than predicted by Figure 6.

**Intermediate Uniform Loading Radius**

The dilatation wave front decays inward from the radius of loading until it is "rounded" to the same condition described for a small radius of loading. This occurs before the dilatation wave reflects from the back face and exhibits the same phenomenon which occurred in the waves generated by a uniform load applied over a small radius.

Unlike the case of the small loading radius, a "hot-spot" of tensile stress develops off the centerline at a position and time indicating the cause is superposition of incident shear and reflected dilatation waves. However, this hot-spot develops late and is smaller than the stress which develops on the centerline.

The stress on the centerline results from the combination of the reflected dilatation wave and the incident wave labeled $S_2$ in Figure 25 (see discussion for large loading radius). It develops on the centerline because the dilatation wave has attenuated away from the centerline. Therefore, the fracture develops on the centerline and propagates outward. A secondary crack may develop off the centerline if the hot-spot has a high enough tensile value.
Large Uniform Loading Radius

The numerical results from a large loading radius clearly support the mathematical analysis of Viswanathan and Biswas [2]. The highest tensile stress develops off the centerline before a high tensile stress develops on the centerline. Also, it develops too early to be caused by combination of the shear wave and reflected dilatation wave. It moves toward the centerline while moving toward the front face. It does not develop until after reflection of the dilatation wave. These conditions mean that the reflected dilatation wave is combining with what Viswanathan and Biswas call "an edge wave . . . with a toroidal front, expanding with velocity $c$," the dilatation wave speed. Figure 25 shows this "edge wave" labeled $S_2$. "Edge" refers to the outer edge of the loaded area on the front face, from which the edge wave emanates. Therefore the crack initiates off the centerline and propagates toward it. The location of the off-centerline tensile stress development is at a radius less than the radius of loading due to the same attenuation "round-off" described earlier.

The stress on the centerline may exceed the tensile strength of the material before the off-centerline crack gets there. A second crack would develop on the centerline and travel outward to meet the first crack in this case.

Finally, it should be noted that attenuation of the wave resulted in the four cases described. If the wave front had maintained the input stress between the centerline
and radius of loading, only the two cases of point load and distributed load need have been discussed. Without attenuation even the smallest of loading radii would produce the highest stress off the centerline first due to superposition of $S_1$ and $S_2$. This would occur near to the back face and at the radius of loading. The shape of the waves does not attenuate or round-off. The magnitude does attenuate from the radius of loading (least attenuation on the centerline), making the waves seem to round-off. The reason for this attenuation may be thought of as a "diffusion" of stress from the highly stressed (saturated) region inside the radius of loading to the region outside the radius of loading where the plate has low stresses. This diffusive process is due to the Poisson effect which causes wave propagation in directions other than the direction of loading.
BIBLIOGRAPHY


APPENDICES
APPENDIX A

DERIVATION OF FINITE ELEMENT MATRICES

Displacement Functions

The displacements within elements are given by:

\[ UR = a_1 + a_2 R + a_3 Z \]  \hspace{1cm} (A.1)
\[ UZ = a_4 + a_5 R + a_6 Z \]

where \( R \) and \( Z \) are the coordinates of the point whose displacement is being calculated. \( UR \) and \( UZ \) are the radial and axial displacements, respectively. Since the displacement functions are valid for the nodes as well as inside the element, the unknown coefficients \( a_1 \) through \( a_6 \) may be determined by writing the displacement functions at each node. Let the nodes and elements of the system be numbered in the manner of Figure 3. Let the nodes of each element be \( i, j \) and \( m \) in a counter-clockwise manner and refer to the global numbers of Figure 3.
$$UR_i = \alpha_1 + \alpha_2 R_i + \alpha_3 Z_i$$

$$UR_j = \alpha_1 + \alpha_2 R_j + \alpha_3 Z_j$$

$$UR_m = \alpha_1 + \alpha_2 R_m + \alpha_3 Z_m$$

$$UZ_i = \alpha_4 + \alpha_5 R_i + \alpha_6 Z_i$$

$$UZ_j = \alpha_4 + \alpha_5 R_j + \alpha_6 Z_j$$

$$UZ_m = \alpha_4 + \alpha_5 R_m + \alpha_6 Z_m$$

The constants $\alpha_1$, $\alpha_2$, and $\alpha_3$ may be solved from the first three equations by Cramer's rule and $\alpha_4$, $\alpha_5$, and $\alpha_6$ are found in the same way from the last three equations. One example and other results follow.

$$\alpha_1 = \frac{\begin{vmatrix}
UR_i & R_i & Z_i \\
UR_j & R_j & Z_j \\
UR_m & R_m & Z_m
\end{vmatrix}}{\begin{vmatrix}
1 & R_i & Z_i \\
1 & R_j & Z_j \\
1 & R_m & Z_m
\end{vmatrix}}$$

(A.3)

Solving A.3 gives:

$$\alpha_1 = \frac{(\alpha_i UR_i + \alpha_j UR_j + \alpha_m UR_m)}{(\alpha_i + \alpha_j + \alpha_m)}$$

(A.4)

where $(\alpha_i + \alpha_j + \alpha_m) = 2A$ = twice the area of the triangle. Similarly the other $\alpha$'s are defined in terms of nodal
displacements. The arithmetic terms \( a, b, \) and \( c \) are defined in Table 2.

\[
\begin{align*}
\alpha_1 &= \frac{(a_i UR_i + a_j UR_j + a_m UR_m)}{2A} \\
\alpha_2 &= \frac{(b_i UR_i + b_j UR_j + b_m UR_m)}{2A} \\
\alpha_3 &= \frac{(c_i UR_i + c_j UR_j + c_m UR_m)}{2A} \\
\alpha_4 &= \frac{(a_i UZ_i + a_j UZ_j + a_m UZ_m)}{2A} \\
\alpha_5 &= \frac{(b_i UZ_i + b_j UZ_j + b_m UZ_m)}{2A} \\
\alpha_6 &= \frac{(c_i UZ_i + c_j UZ_j + c_m UZ_m)}{2A}
\end{align*}
\]

(A.5)

**TABLE 2**

COEFFICIENTS USED IN THE DEFINITION OF DISPLACEMENT

| \( a_i = (R_j Z_m - R_m Z_j) \) | \( b_i = (Z_j - Z_m) \) | \( c_i = (R_m - R_j) \) |
| \( a_j = (R_m Z_i - R_i Z_m) \) | \( b_j = (Z_m - Z_i) \) | \( c_j = (R_i - R_m) \) |
| \( a_m = (R_i Z_j - R_j Z_i) \) | \( b_m = (Z_i - Z_j) \) | \( c_m = (R_j - R_i) \) |

Strain-Displacement Relations

The definition of strain in terms of displacement for axisymmetric problems is:
\[
\begin{align*}
\varepsilon_R &= \frac{3\varepsilon_{UR}}{\varphi R} = a_2 \\
\varepsilon_\theta &= \frac{\varepsilon_{UR}}{R} = \frac{a_1}{RBAR} + a_2 + \frac{a_3ZBAR}{RBAR} \\
\varepsilon_Z &= \frac{3\varepsilon_{UZ}}{\varphi Z} = a_6 \\
\gamma_{RZ} &= \frac{3\varepsilon_{UR}}{\varphi Z} + \frac{3\varepsilon_{UZ}}{\varphi R} = a_3 + a_5
\end{align*}
\]  
(A.6)

The definition of \( \varepsilon_\theta \) includes the following approximation since these are constant strain elements.

\[
\begin{align*}
RBAR &= \frac{(R_i + R_j + R_m)}{3} \\
ZBAR &= \frac{(Z_i + Z_j + Z_m)}{3}
\end{align*}
\]  
(A.7)

Define the element displacement matrix as:

\[
\{\delta\}_e = \begin{bmatrix}
UR_i \\
UZ_i \\
UR_j \\
UZ_j \\
UR_m \\
UZ_m
\end{bmatrix}
\]  
(A.8)

Substitution of the definitions of the constants, \( a_i \), results in the strain displacement equations in matrix form. Equations A.6 become:
\begin{equation}
\begin{bmatrix}
\epsilon_R \\
\epsilon_\theta \\
c_Z \\
\gamma_{RZ}
\end{bmatrix} = \frac{1}{2A} \begin{bmatrix}
b_1 & 0 & b_j & 0 & b_m & 0 \\
d_1 & 0 & d_j & 0 & d_m & 0 \\
0 & c_i & 0 & c_j & 0 & c_m \\
c_i & b_i & c_j & b_j & c_m & b_m
\end{bmatrix} \{\delta\} \quad (A.9)
\end{equation}

where:

\begin{align*}
d_i &= \frac{a_i}{RBAR} + b_i + \frac{c_iZBAR}{RBAR} \\
d_j &= \frac{a_j}{RBAR} + b_j + \frac{c_jZBAR}{RBAR} \\
d_m &= \frac{a_m}{RBAR} + b_m + \frac{c_mZBAR}{RBAR}
\end{align*} \quad (A.10)

**The Stiffness Matrix**

The element stiffness was derived on page 10 to be:

\[ [K]_e = 2 \cdot \pi \cdot RBAR \cdot A[B]^T[D][B] \quad (A.11) \]

In order to develop a procedure for assembling the stiffness matrix for many elements, consider a two element example, shown in Figure 28.

For the system, equilibrium of nodal forces gives:
Figure 28. Two Element Example
\[ S_1(1R) = 0 \]
\[ S_1(2Z) = R_{1Z} \]
\[ S_1(2R) + S_2(2R) = 0 \]
\[ S_1(2Z) - S_2(2Z) = R_{2Z} \]
\[ S_2(3R) = 0 \]
\[ S_2(3Z) = Q_{3Z} \]
\[ S_2(4R) + S_2(4R) = 0 \]
\[ S_2(4Z) + S_2(4Z) = Q_{4Z} \]

Notation: \( S_i(1R) \)

The element stiffness equilibrium equations define the nodal forces \( (S) \) in terms of stiffness times displacement. Substitution of these terms in the above system equilibrium equations results in the assembled stiffness matrix for the two element system. In order to show that the position of the element stiffness term in the assembled stiffness matrix is given by node number, the element stiffness matrices for the two element example are given in the equations below in expanded form. The program in Appendix B uses the same minimum space (KELM) for each element stiffness matrix and in the subroutine KFORM computes the positions to add them to in the assembled stiffness matrix (KASY) by node number and a pointer matrix (NADJ). Maximum
storage compression is achieved by using NADJ to eliminate the zero terms in a row, resulting in KASY having only fourteen columns.

\[ [K]_1 \{\delta\} + [K]_2 \{\delta\} = \{R\} \quad (A.13) \]

\[
[K]_1 = \begin{bmatrix}
  k_{11} & k_{12} & 0 & k_{13} \\
  k_{21} & k_{22} & 0 & k_{23} \\
  0 & 0 & 0 & 0 \\
  k_{31} & k_{32} & 0 & k_{33}
\end{bmatrix} \quad (A.14)
\]

\[
[K]_2 = \begin{bmatrix}
  0 & 0 & 0 & 0 \\
  0 & k_{11} & k_{12} & k_{13} \\
  0 & k_{21} & k_{23} & k_{24} \\
  0 & k_{31} & k_{32} & k_{33}
\end{bmatrix} \quad (A.15)
\]
Each \( k \) in the above expanded element stiffness matrices (A.14 and A.15) is a 2 by 2 partition of the original 6 by 6 element stiffness matrix. The subscript refers to the row and column position of the partition in the 6 by 6 element stiffness matrix. The superscript is determined by the \( i, j, m \) node numbers of the element and is the position of the partition in the assembled stiffness matrix. This is further illustrated below. Consider any one element with some external loading (R).

\[
[K]_e \{\delta\}_e = \{R\} \tag{A.17}
\]

\[
\begin{bmatrix}
1 & 1 & k_{11} & k_{12} & k_{13} \\
2 & j & k_{21} & k_{22} & k_{23} \\
3 & m & k_{31} & k_{32} & k_{33}
\end{bmatrix}
\begin{bmatrix}
\delta_1 \\
\delta_j \\
\delta_m
\end{bmatrix}
= 
\begin{bmatrix}
R_1 \\
R_j \\
R_m
\end{bmatrix} \tag{A.18}
\]
where $\delta_1$, $R_1$, etc., are 2 by 1 partitions such that:

$$
\delta_1 = \begin{cases} 
UR_1 \\
UZ_1 
\end{cases}
$$

and so forth.

The system of forcing KELM to conform to the $i, j, m$ sequence of nodal displacements in the $\{\delta_e\}$ array insures that the proper indexing of the partition will occur.

The Lumped Mass Matrix

The lumped mass matrix is computed by the subroutine MFORM. Since the grid generated by this program is regular and defined by the number of nodes axially and radially, the mass matrix is formed node by node. The nature of the grid (Figure 3) allows the matrix to be computed in stages, as indicated by the comment cards (see Appendix C). For example, all interior nodes are shared by six elements, so the factor $CF$ is equal to 2. The average of the radii to the centroids ($RBAR's$) of the six elements surrounding any interior node is always the same as the radius of the node, so $DR = 0$. Similarly the mass lumped at each node is based on the number of elements which share the node and the average of the radii to the centroids of those elements. The lumped mass matrix for the two element example in Figure 28 is:
\[
[M] = \begin{bmatrix}
0.333 \rho \bar{A}c_1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0.333 \rho \bar{A}c_1 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0.667 \rho \bar{A}c_2 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0.667 \rho \bar{A}c_2 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0.333 \rho \bar{A}c_3 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0.333 \rho \bar{A}c_3 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0.667 \rho \bar{A}c_2 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0.667 \rho \bar{A}c_2 \\
\end{bmatrix}
\]

where,

\( c_1 = \text{RBAR for element 1} \)

\( c_2 = R_{1Z} - R_{2Z} \)

\( c_3 = \text{RBAR for element 2}. \)
APPENDIX B

DERIVATION OF LUMPED AND CONSISTENT MASS MATRICES FOR ONE DIMENSIONAL PROBLEM WITH LAPLACE TRANSFORM SOLUTION

Description of Problem

In order to determine which mass matrix approach should be used for propagation problems, both will be used to generate equations approximating a bar subjected to a step tensile load at the free end and fixed at the other end. Laplace transformation will be the method of solution of the equations resulting in an expression for the fixed end reaction at \( t = 0^+ \). One-dimensional finite elements are used to approximate the bar which is assumed to have a length many times greater than the diameter.

Lumped Mass Matrix
The subscript refers to the node and the superscript refers to the element. Let an element be described in general as follows and choose a linear displacement function.

\[
A = \text{area} \\
E = \text{Young's modulus} \\
\rho = \text{mass density}
\]

\[
u = a_1 + a_2 x
\]  
(B.1)

\[
\varepsilon_x = a_2 = \frac{u_j - u_i}{\ell}
\]  
(B.2)

\[
[B] = \frac{1}{\ell} \begin{bmatrix} -1 & 1 \end{bmatrix}
\]
(B.3)

\[
[D] = [E]
\]
(B.4)

\[
\{\delta\}_e = \begin{bmatrix} u_1 \\ u_j \\ u_3 \end{bmatrix}, \{\delta\} = \begin{bmatrix} u_1 \\ u_2 \\ u_3 \end{bmatrix} = \begin{bmatrix} u_1 \\ \vdots \\ u_3 \end{bmatrix}
\]
(B.5)

\[
[K]_e = A \varepsilon [B]^T[D][B] 
\]
(B.6)

\[
[K]_e = \frac{AE}{\ell} \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix}
\]
(B.7)

\[
\{\delta\}_e = [K]_e \{\delta\}_e
\]
(B.8)
\[ S_1^1 = R_1 \]
\[ S_2^1 + S_2^2 = 0 \quad \text{(B.9)} \]
\[ S_3^2 = Q \]

\[
\frac{\mathbf{AE}}{\mathbf{\ell}} \begin{bmatrix}
    1 & -1 & 0 \\
    -1 & 2 & -1 \\
    0 & -1 & 1 \\
\end{bmatrix}
\begin{bmatrix}
    u_1 \\
    u_2 \\
    u_3 \\
\end{bmatrix} = 
\begin{bmatrix}
    R_1 \\
    0 \\
    Q \\
\end{bmatrix} \quad \text{(B.10)}
\]

\[
[M] = \begin{bmatrix}
    1/2 & 0 & 0 \\
    0 & 1 & 0 \\
    0 & 0 & 1/2 \\
\end{bmatrix},
\quad m_o = \rho A \ell = \text{mass of one element} \quad \text{(B.11)}
\]

\[
[M]\ddot{\mathbf{\delta}} + [K]\mathbf{\delta} = \{R\} \quad \text{(B.12)}
\]

Let \( Q(t) = QH(t) \).

By following the pattern shown for two elements, the equations for four elements may be written:
\[
\begin{bmatrix}
\frac{1}{2} & 0 & 0 & 0 & 0 \\
0 & 1 & 0 & 0 & 0 \\
0 & 0 & 1 & 0 & 0 \\
0 & 0 & 0 & 1 & 0 \\
0 & 0 & 0 & 0 & \frac{1}{2}
\end{bmatrix}
\begin{bmatrix}
u_1 \\
u_2 \\
u_3 \\
u_4 \\
u_5
\end{bmatrix}
= 
\begin{bmatrix} 1 & -1 & 0 & 0 & 0 \\
-1 & 2 & -1 & 0 & 0 \\
0 & -1 & 2 & -1 & 0 \\
0 & 0 & -1 & 2 & -1 \\
0 & 0 & 0 & -1 & 1
\end{bmatrix}
\begin{bmatrix} u_1 \\
u_2 \\
u_3 \\
u_4 \\
u_5
\end{bmatrix}
\]

\[
\begin{cases}
R_1 \\
0 \\
0 \\
0 \\
Q(t)
\end{cases}
\]

(B.13)

Divide both sides by \( \frac{m_0}{2} \), recall \( m_0 = \rho A l \), and longitudinal wave speed, \( c = \sqrt{E/\rho} \).

\[
\ddot{u}_1 + \left(\frac{c}{\ell}\right)^2 (2u_1 - 2u_2) = \frac{2R_1}{m_0}
\]

\[
2\ddot{u}_2 + \left(\frac{c}{\ell}\right)^2 (-2u_1 + 4u_2 - 2u_3) = 0
\]

\[
2\ddot{u}_3 + \left(\frac{c}{\ell}\right)^2 (-2u_2 + 4u_4 - 2u_4) = 0
\]

\[
2\ddot{u}_4 + \left(\frac{c}{\ell}\right)^2 (-2u_3 + 4u_4 - 2u_5) = 0
\]

\[
\ddot{u}_5 + \left(\frac{c}{\ell}\right)^2 (-2u_4 + 2u_5) = \frac{2QH(t)}{m_0}
\]

(B.14)

Initially, at \( t = 0 \), the displacements and velocities of all five nodes are zero. The boundary condition exists such that the left end is fixed, or, the displacement,
velocity and acceleration of node one are zero for all time. This boundary condition means that the Laplace transforms of $u_1$ and $\ddot{u}_1$, are also zero. The Laplace transforms of the equations with the initial and boundary conditions substituted are:

$$-\left(\frac{c}{\lambda}\right)^2 U_2 = \frac{L[R_1]}{m_0}$$

$$s^2 U_2 + \left(\frac{c}{\lambda}\right)^2 (2U_2 - U_3) = 0$$

$$s^2 U_3 + \left(\frac{c}{\lambda}\right)^2 (-U_2 + 2U_4 - U_4) = 0 \quad (B.15)$$

$$s^2 U_4 + \left(\frac{c}{\lambda}\right)^2 (-U_3 + 2U_4 - U_5) = 0$$

$$s^2 U_5 + \left(\frac{c}{\lambda}\right)^2 (-2U_4 + 2U_5) = \frac{2Q}{sm_0}$$

where the capital $U$ indicates the Laplace transform of the displacement and $s$ is the Laplace operator.

It is clear from the first equation that $U_2$ can be expressed as a constant times the transform of $R_1$. This is substituted for $U_2$ in the succeeding equations. Then, with that substitution in the second equation, $U_3$ is expressed in terms of the transform of $R_1$. The substitutions continue and after substitution and rearrangement the fourth equation expresses $U_5$ in terms of the transform of $R_1$. The fifth equation is used to find the transform of $R_1$ in terms of the transform of the forcing function. The result is separated by partial fractions into recognizable transforms which are the transforms
of sine functions and the transform of the forcing function all multiplied by constants. The inverse of this Laplace expression is the reaction, \( R_1 \), as a function of time. At time zero, the four sine terms vanish, and

\[ R_1 \bigg|_{t=0} = -\frac{9}{5}. \] (B.16)

Additional elements result in more terms in the series and \( R_1 \) at time zero approaches zero.

**Consistent Mass Matrix**

The mass of the element is considered to be distributed. Its inertia is assumed to act as a body force causing nodal forces, \( S^b \), in combination with the nodal forces, \( S^e \), caused by the stiffness of the element. The virtual work expression must then include the contribution of the mass inertia.

Let \( X = \rho \ddot{u} = \) body force.

\[ S^1_{ii} u_i + S^1_{ij} u_j = \int_{x_1}^{x_j} \sigma \varepsilon \text{d}x + \int_{x_1}^{x_j} X u \text{d}x \] (B.17)

The displacement function is now variable with position and time.

\[ u(x,t) = a_1 + a_2 x(t) \] (B.18)

\[ u(x,t) = u_j \left[ \frac{x - x_i}{\ell} \right] - u_i \left[ \frac{x_i - x}{\ell} \right] \] (B.19)
\( \ddot{u}(x,t) = \ddot{u}_j \left[ \frac{x - x_1}{\ell} \right] - \dot{u}_1 \left[ \frac{x_1 - x}{\ell} \right] \) \hspace{1cm} (B.20)

rewriting B.17

\[
(S^b + S^e)_1^i u_1 + (S^b + S^e)_1^j u_j = \int_{x_1}^{X_j} \sigma \varepsilon A dx + \int_{x_1}^{X_j} X u A dx
\] \hspace{1cm} (B.21)

The contribution of the inertia forces may be formed by separating the body force terms.

\[
(S^b)_1^i u_1 + (S^b)_1^j u_j = \frac{1}{\ell} \int_{x_1}^{X_j} X A [u_j(x - x_1) - u_1(x - x_j)] dx
\] \hspace{1cm} (B.22)

Equating coefficient of \( u_1 \) and \( u_j \);

\[
(S^b)_1^i = \frac{1}{\ell} \int_{x_1}^{X_j} X A (x_j - x) dx
\]

\[
(S^b)_1^j = \frac{1}{\ell} \int_{x_1}^{X_j} X A (x - x_1) dx
\] \hspace{1cm} (B.23)

Substitution of \( X = \rho \ddot{u} \),

\[
x_1 = (1 - i) \ell,
\]

\[
x_j = i \ell,
\]

rearranging, substituting \( Z = \left( \frac{X}{\ell} - i \right) \), and with the corresponding changes in the limits of integration, the nodal forces due to inertia forces may be written;
\[(S^b)^i_1 = -m_o \int_{-1}^{0} ((\dddot{u}_j - \dddot{u}_1)z^2 + \ddot{u}_j z)dz \quad \text{(B.24)}\]

\[(S^b)^i_j = -(S^b)^i_1 - m_o \int_{-1}^{0} ((\dddot{u}_j - \dddot{u}_1)z + \ddot{u}_j)dz \]

Evaluation of the integrals yields;

\[(S^b)^i_1 = m_o \left[ \frac{u_j}{6} + \frac{\ddot{u}_1}{3} \right] \quad \text{(B.25)}\]

\[(S^b)^i_j = m_o \left[ \frac{u_j}{3} + \frac{\ddot{u}_1}{6} \right] \]

By writing equilibrium of forces at the nodes of the two element problem, remembering that the nodal forces consist of both inertia and elastic forces, the following set of equations is formulated.

\[
m_o \begin{bmatrix}
\frac{1}{3} & \frac{1}{6} & 0 \\
\frac{1}{6} & \frac{2}{3} & \frac{1}{6} \\
0 & \frac{1}{6} & \frac{1}{3}
\end{bmatrix}
\begin{bmatrix}
\dddot{u}_1 \\
\dddot{u}_2 \\
\dddot{u}_3
\end{bmatrix}
\]

\[
+ \frac{AE}{I} \begin{bmatrix}
1 & -1 & 0 \\
-1 & 2 & -1 \\
0 & -1 & 1
\end{bmatrix}
\begin{bmatrix}
\dddot{u}_1 \\
\dddot{u}_2 \\
\dddot{u}_3
\end{bmatrix}
= \begin{bmatrix}
R_1 \\
0 \\
Q
\end{bmatrix}
\quad \text{(B.26)}
\]

where the first matrix is the so-called consistent mass matrix and the stiffness matrix is the same as before. 
\(M_o + K_o\) give the nodal forces.
After formulating the four element problem with the consistent mass matrix, the solution of the five resulting equations by Laplace transform follows the same procedure as shown for the four element lumped mass problem. The same initial and boundary conditions are used and the reaction at node one is found by successive substitution. Again, the solution is a step plus sine functions all multiplied by constants. At time zero, only the step acts on node one and the constant coefficient is unity. The reaction at \( t = 0(+) \) is:

\[
R_1 \bigg|_{t=+0} = -Q
\]  

(B.27)

Even with additional elements, \( R_1 = -Q \) at time zero. Therefore, the consistent mass matrix does not allow propagation of transient phenomenon. The reaction, \( R_1 \), is a Fourier sine series in which the first term is a constant since the forcing function is a unit step. It is well known that Fourier series solutions for a step, or any discontinuous function, have a jump at the discontinuity known as Gibbs phenomenon and proportional to the size of the discontinuity (See Biot and von Kármán [9]). Further, the above analyses indicate that the dynamic finite elements solutions are equivalent to Fourier series solutions even though the process is numerical in the computer. Therefore, the oscillations in the finite element solution are
due to too few terms in the series (too few nodes) and Gibb's phenomenon.

Finally, it is easy to see that the equations of motion developed by lumping the mass at the nodes are the same as the one dimensional wave equation. The comparison is obvious when the accelerations in the finite element method are written in finite difference notation. Then the differential equation for one-dimensional waves is written in finite difference notation. The finite element stiffness times displacement terms are already equivalent to the difference notation. However, when the accelerations in the finite element equations for the consistent mass matrix are written in difference form, the result does not correspond to the difference form of the one dimensional wave equation. This qualitative look at the nature of the equations further supports the idea that the consistent mass matrix will not allow propagation.
APPENDIX C
APPENDIX C

COMPUTER PROGRAM JMMSPALL

Following is a listing of the program used in this research. A sample printout is included, consisting of a typical printout for one time step. In use, the program solves a dynamic axisymmetric problem and prints out stresses and displacements at several times. The sample problem is a plate with a distributed load. There are twenty-four nodes axially and thirty-five nodes radially. Distributed loads are produced by resolving the desired stress to nodal forces.
CALL DATSUB(DT,TDT,NNODE,NFS,NF,NELM,A,DL,NZZ)
CALL SOLVE(NNODE,DT,NFS,NF,TDT,T,NELM,A,DL,NZZ)
WRITE(6,801)
801 FORMAT(///,30X,'T=',E10.4,'PROGRAM SPALL COMPLETE FOR SPECIFIED
1TIME OF PROPAGATION')
STOP
END
SUBROUTINE DATSUB(DT,TDT,NNOD,NFS,NF,NELM,A,DL,NZZ)
DIMENSION DB(24)
COMMON B(24),KELM(36),KASY(1680,14),MASS(1680),R(840),Z(840),
IF(45),D(16),NI(1564),NJ(1564),NM(1564),NADJ(840,7),NZRU(560)
INTEGER N,NJ,NM,NADJ,NZRU
REAL KASY,KELM,MASS
READ(5,1000)DT,E,PR,RHO
1000 FORMAT(4E10.4)
READ(5,1004)NF,NELM,NNOD,NCL,NFC,NZZ,NZR
1004 FORMAT(7I4)
CMU=E/(2.*(1.+PR))
CLMBA=PR*E/(1.+PR)*(1.-2.*PR)
VDIL=SQR((CLMBA+2.*CMU)*386.4/RHO)
VSHFAR=SQR((386.4*CMU/RHO)
DT=.50*DL/VDIL
TDT=76*DT
H=(NCL+1)*D:
WRITE(6,1001)DT,D,TDT,DL,H,E,PR,RHO
1001 FORMAT(1H1,/,10X,'TIME INCREMENT',DT='E11.5',/10X,'TOTAL TIME ',
1'OF PROPAGATION='Z11.5',/10X,'ELEMENT SIZE='E11.5',/10X,
2'PLATE THICKNESS='E11.5',/10X,'YOUNGS MODULUS='E11.5',/10X,
3'POISSONS RATIO='E11.5',/10X,'WEIGHT DE mystery)'E11.5)
WRITE(6,1008)VDIL,VSHFAR
1008 FORMAT(//10X,'DILITATION WAVE SPEED='E12.5,'INCHES PER SEC',//,
110X,'DISTORTIONAL WAVE SPEED='E12.5,'INCHES PER SEC',//10X,
2'D MATRIX',/)
DO 9988 I=1,16
9988 D(I)=0.0
D(1)=1.-PR
D(2)=PR
D(3)=PR
D(5)=PR
D(6)=1.*=PR
D(7)=PR
D(9)=PR
D(10)=PR
D(11)=1.*=PR
D(16)=.5*(1.*=2.*PR)
DO 1002 I=1,4
1002 WRITE(6,1003) (D(J),J=1,16,4)
1003 FORMAT(4F10.4)
READ(5,1005) (F(I),I=1,NF)
1005 FORMAT(7L11.5)
P=F(I)
PI=3.1416
NF=24
NFS=NFC=NF+3
N=NFC=1
DO 9999 IFF=1,N
9999 F(IFF)=2.*(NF=IFF)*DF*D*L*P*PI
F(NF)=P*PI*D*L*DEF*.25
WRITE(6,1104)NF,NELM,NNOD,NCL,NFC,NZZ,NZR
1104 FORMAT(//,1X,1 NF NELM NNOD NCL NFC NZZ NZR,/,1X,7I5)
WRITE(6,1006) (F(I),I=1,NF)
1006 FORMAT(//,10X,'FORCLES',/,10(2X,L11.5))
ITOP=NZZ=1
ISUM=0
DO 1 I=1,ITOP,2
IBASE=NFC+2
INC=2*IBASE
ISUM=ISUM+INC
NZRU(I)=IBASE+ISUM=INC
1 NZRU(I+1)=NZRU(I)+1
ISUM=0
ISTART=NZ+1
ITOP=NZR=1
DO 2 I=ISTART,ITOP,2
IBASE=1
ISUM=ISUM+INC
NZRU(I)=IBASE+ISUM=INC
2 NZRU(I+1)=NZRU(I)+INC=IBASE
WRITE(6,1107)(NZRU(I),I=1,NZR)
1107 FORMAT('///',5X,'NZRU',Z(/,IX,2414))
IN=0
CZ=1.0
5 CZ=CZ+1.0
CR=NFC+2.
ICR=CR
DO 12 I=1,ICR
CR=CR=1.0
IN=IN+1
R(IN)=CR
12 Z(IN)=CZ
CR=1.0
CZ=CZ+1.0
DO 20 I=1,ICR
CR=CR+1.0
IN=IN+1
R(IN)=CR
20 Z(IN)=CZ
IF(CZ=NELM/(Z*(NFC+1)))5,25,25
25 DO 30 N=1,NNOD
   R(N)=R(N)*DZ
30 Z(N)=Z(N)*DC
    J=1
    K=NFC+1
    LL=1
    IT=INC
32 L=LL
   II=0
   DO 40 I=J,K
   II=II+1
   IM=IT=II+1
   NI(L)=I
   NJ(L)=I+1
   NM(L)=IM
   L=L+NELM/(Z*(NFC+1))
   NI(L)=I+1
   NJ(L)=IM+1
   NM(L)=IM
40 L=L+NELM/(Z*(NFC+1))
   IF(J=NNOD+INC=1)41,51,51
41 J=J+INC
   K=J+NFC
   IT=IT+INC
   LL=LL+2
   GO TO 32
51 J=INC
   K=J+NFC
   IT=INC+1
60  L=LL
   II=0
   DO 70  I=J,K
   II=II+1
   IJ=IJ+1
   NI(L)=IJ+1
   NJ(L)=IJ
   NM(L)=1+1
   L=L+NELM/(2+(NFC+1))
   NI(L)=IJ
   NJ(L)=1+2
   NM(L)=1+1
70  L=L+NELM/(2+(NFC+1))
    IF(J=NNOD+INC)71,72,72
71  J=J+INC
    K=J+NFC
    IT=IT+INC
    LL=LL+2
    GO TO 60
72  CONTINUE
69  N=1,NNOD
60  K=1,7
91  NADJ(N,K)=0
    IC34=NFC+1
   NCLB=NNOD-NFC+1
   NCLF=NFC+2
   NADJ(1,1)=1
   NADJ(1,2)=2
   NADJ(1,3)=INC
NADJ(3*NCLF,5)=3*NCLF+2
KK=1
N=NFC+3
NTOP=(NZZ=2)/2=1
77 DO 79 I=1,NTOP
    DO 78 K=1,5
75 NADJ(N+INC,K)=NADJ(N,K)+INC
79 N=N+INC
    KK=KK+1
    IF(KK=2)82,82,83
82 N=INC
83 IF(KK=3)77,84,85
84 N=INC+1
85 IF(KK=4)77,86,87
86 N=3*NCLF
GO TO 77
87 DO 104 N=2,IC34
    NADJ(N,1)=N=1
    NADJ(N,2)=N
    NADJ(N,3)=N+1
    NADJ(N,4)=INC+1-N
104 NADJ(N,5)=INC+2=N
    IV=NNO=1
    ITV=NNO=1
DO 105 N=1V,ITV
    NADJ(N,1)=NCLB=N
    NADJ(N,2)=IV=N
    NADJ(N,3)=N-1
    NADJ(N,4)=N
105 NADJ(N,5)=N+1
I=IC34+3
J=INC-1
K=1
102 I1=I-3
I1=0
I7=I+2*IC34-1
106 DO 110 N=I,J
  NADJ(N,1)=I1+II
  NADJ(N,2)=I1+II+1
  NADJ(N,3)=N-1
  NADJ(N,4)=N
  NADJ(N,5)=N+1
  NADJ(N,6)=17-II-1
  NADJ(N,7)=17-II
110 II=II+1
  IF(K=2)111,130,130
111 IF(I=NNOD+INC+NF-1)112,120,120
112 I=I+INC
  J=I+NF-1
  GO TO 102
120 K=2
  I=2
130 IF(I=NNOD+INC-2)131,140,140
131 I=I+INC
  J=I+NF-1
  I1=I-4
  I1=0
  I7=I+2*IC34
  GO TO 106
140 CONTINUE
C = E / ((1 + PR) * (1 - 2 * PF))
DO 10 I = 1, 16
10 DI(I) = D(I) * C
NRANGE = 2 * NNCD
100 DO 103 I = 1, NRANGE
    MASS(I) = 0.0
    DO 103 K = 1, 14
103 KASY(I, K) = 0.0
    DO 101 I = 1, 24
101 DB(I) = 0.0
11 DO 50 L = 1, NLM
    CALL BFORM(L, RBAR, A)
    CALL JMPRD(C, B, DB, 4, 4, 6)
    IR = 1
    DO 90 K = 1, 6
    DO 90 J = 1, 6
        KELM(IR) = 0.0
    DO 80 I = 1, 4
        IA = 4 * (J - 1) + 1
        IB = 4 * (K - 1) + 1
80 KELM(IR) = KELM(IR) + B(IA) * DB(IB)
90 IR = IR + 1
A = A
DO 49 I = 1, 36
49 KELM(I) = 6.25 * RBAR * A * KELM(I)
    CALL KFORM(L)
50 CONTINUE
    CALL MFORM(DT, A, DE, NFC, NNCD, NZ, NCI, NZR, RHO)
RLTUR
END
SUBROUTINE BFORM(L,RBAR,A)
COMMON B(24),KELM(36),KASY(1680,14),MASS(1680),R(840),Z(840),
1F(45),D(16),NJ(1564),NJ(1564),NM(1564),NADJ(840,7),NZRU(560)
INTEGER*2 NI,NJ,NM,NADJ,NZRU
RBAR=(R(NI(L))+(NJ(L))+(NM(L)))/3.0
ZBAR=(Z(NI(L))+Z(NJ(L))+Z(NM(L)))/3.0
DO 10 I=1,24
10 B(I)=0.0
A=0.5*(R(NJ(L))Z(NM(L)))+R(NM(L))Z(NJ(L))+R(NM(L))Z(NI(L))-
1R(NJ(L))Z(NM(L))R(NI(L))Z(NJ(L))R(NJ(L))Z(NI(L))
B(1)=(Z(NJ(L))+Z(NM(L)))/(2.*A)
B(4)=(R(NM(L))+R(NJ(L)))/(2.*A)
B(2)=(R(NJ(L))+Z(NM(L))+R(NM(L)+Z(NJ(L)))/(RBAR*2.*A)
1*B(1)+B(4)*ZBAR/RBAR
B(7)=B(4)
B(8)=B(1)
B(9)=(Z(NM(L))+Z(NI(L)))/(2.*A)
B(12)=(R(NI(L))+R(NM(L)))/(2.*A)
B(10)=(R(NM(L))+Z(NI(L))+R(NI(L))Z(NM(L)))/(RBAR*2.*A)
1*B(9)+B(12)*ZBAR/RBAR
B(15)=B(12)
B(16)=B(9)
B(17)=(Z(NI(L))+Z(NJ(L)))/(2.*A)
B(20)=(R(NJ(L))+R(NM(L)))/(2.*A)
B(18)=(R(NI(L))+Z(NJ(L))+R(NJ(L))Z(NI(L)))/(RBAR*2.*A)
1*B(17)+B(20)*ZBAR/RBAR
B(23)=B(20)
B(24)=B(17)
RETURN
END
SUBROUTINE KFORM(L)
DIMENSION JORD(3)
COMMON B(24), KELM(36), KASY(1680, 14), MASS(1680), R(840), Z(840), 
1F(45), D(16), NI(1564), NJ(1564), NM(1564), NADJ(840, 7), NZRU(560)
INTEGER*2 NI, NJ, NM, NADJ, NZRU
REAL KELM, KASY
N=NI(L)
IT=1
ICR=1
IR=2*N=1
2 NC=NI(L)
I=1
3 DO 5 K=1, 7
 IF NC=NADJ(N, K)) 990, 7, 5
5 CONTINUE
7 JORD(I)=K
9 NC=NJ(L)
 I=I+1
 IF (I=2) 9, 3, 10
10 NC=NML(L)
 IF (I=3) 9, 3, 11
11 ICC=1
12 JC=2*JORD(ICC)=1
 KASY(IR, JC)=KASY(IR, JC)+KELM(IT)
 JC=2*JORD(ICC)
 KASY(IR, JC)=KASY(IR, JC)+KELM(IT+6)
 ICC=ICC+1
 IT=IT+12
 IF (ICC=3) 12, 12, 13
13 ICR=ICR+1
IR = IR + 1
IT = ICR
IF (ICR = 3) 11, 20, 29
20 N = NJ(L)
   IR = 2 * N - 1
29 IF (ICR = 5) 2, 30, 39
30 N = NM(L)
   IR = 2 * N - 1
39 IF (ICR = 6) 2, 11, 40
990 WRITE (6, 991) N
991 FORMAT (1H1, //, 30X, 'PROGRAM SPALL TERMINATED DUE TO INCORRECT ',
           'NADJ DATA FOR NODE N=', 14)
      STOP
40 RETURN
END
SUBROUTINE MFORM(DT,A,DE,NFC,NNOD,NZZ,NCL,NZR,RHO)
COMMON B(24),KFLM(36),KASY(1680,14),MASS(1680),R(840),Z(840),
   IF(45),D(16),NI(1564),NJ(1564),NM(1564),NADJ(840,7),NZRU(560)
INTEGER*2 NI,NJ,NM,NADJ,NZRU
REAL M,MASS
NS=NNOD-NFC+3
K=0
II=1
MASS(1)=0.333*6.284*RHO*(R(1)=0.333*DE)*A/386.4
MASS(2)=MASS(1)
CF=1.0
III=NZZ+1
C FRONT FACE NODES
   DR=2.*DE/9.
   I=2
   J=NFC+1
58 DO 60 N=I,J
   IRM=2*N-1
   M=CF*6.284*RHO*(R(N)+DR)*A/386.4
   MASS(IRM)=M
   MASS(IRM+1)=M
60 CONTINUE
C INTERIOR NODES
61 I=N+3
   CF=2.0
   J=I+NFC+1
   DR=0.0
   IF(J(NS)=58,58,62
C BACK FACE NODES
62 DR=2.*DE/9.0
CF=1.0
IF(I=NNOD)58,63,63
63 K=1
C NODES
DR=4.*DE/9.0
CF=1.0
II=II+1
N=NZRU(II)
IF(II=(NCL+1))59,59,64
64 K=2
C NODES
DR=4.*DE/9.0
II=II+1
N=NZRU(II)
IF(II=NZR)59,65,65
65 K=K+1
IF(K=3)65,66,67
C NODE ON FRONT FACE, CL
66 CF=2.*3.
DR=DE/2.0
N=NFC+2
GO TO 59
C NODE ON BACK FACE, CL
67 CF=1.*3.
DR=DE/3.
N=NNOD-NFC+1
IF(K=4)66,59,68
59 IRM=2*N+1
M=CF*6.284*RHO*(R(N)+DR)*A/386.4
MASS(IRM) = M
MASS(IRM+1) = M
IF (K=1) 63, 63, 601
601 IF (K=2) 64, 64, 65
68 RETURN
END
SUBROUTINE SOLVE(NNOD,DT,NFS,NF,TDT,T,NELM,AE,DE,NZZ)
DIMENSION U(ELM(6),STRAIN(4),STRESS(4),U(1680),UM1(1680),UP1(1680),
IANS(2,25)
COMMON B(24),KELM(36),KASY(1680,14),MASS(1680),R(840),Z(840),
IF(45),D(16),NI(1564),NJ(1564),NM(1564),NADJ(840,7),NZRU(560)
INTEGER*2 NI,NJ,NM,NADJ,NZRU
REAL MASS,KASY
IFF=0
ANS(2,1)=0.0
PI=3.14159
NCL=NZZ=2
TTF=45*DT
TPR=76*DT
T=0.0
NN=NNOD*2
DO 200 I=1,NN
UM1(I)=0.0
U(I)=0.0
200 UP1(I)=0.0
NN=NNOD=1
NNN=2*NN
DO 250 N=1,NN
UP1(N)=0.0
IF(N=NFS)205,201,201
201 IFF=IFF+1
202 IF(IFF=NFS)202,202,205
202 FORCE=F(IFF)*0.01
WFORCE=FORCE
GO TO 206
205 FORCE=0.0
206 IRR=2*N-1
   IRZ=IRR+1
   SUMKUR=0.0
   SUMKUZ=0.0
   DO 210 J=1,7
207 IF(NADJ(N,J))249,249,207
   IC=2*J
   IUR=2*NADJ(N,J)+1
   IUZ=IUR+1
   SUMKUR=SUMKUR*KASY(IRR,IC-1)*U(IUR)+KASY(IRR,IC)*U(IUZ)
   SUMKUZ=SUMKUZ+KASY(IRZ,IC-1)*U(IUR)+KASY(IRZ,IC)*U(IUZ)
210 UP1(IRR)=U(IRR)-DT*DT*SUMKUR/(2.0*MASS(IRR))
249 UP1(IRZ)=U(IRZ)-DT*DT*(FORCE-SUMKUZ)/(2.0*MASS(IRZ))
250 NSTEP=2*(NCL+1)
259 CONTINUE
   U(1)=0.0
   NSTART=1
   NTOP=NELM=2*(NCL+1)+1
   IFF=0
   DO 260 I=1,NNN
      UM1(I)=U(I)
260 U(I)=UP1(I)
   WRITE(6,262)T,WFORCE
262 FORMAT(/,25X,'T=',E12.5,5X,'FORCE=',E12.5)
   DO 263 NR=1,NZZ
      IU=2*NZRU(NR)+1
   263 U(IU)=0.0
      IF(NLT.TPR) GO TO 703
   WRITE(6,220)T
220 FORMAT(1H1,40X,STRESS FIELD BY AVERAGING ADJACENT','
1*ELEMENTS*,///,59X,*R&E2 COMPONENTS*,10X,*T=*,E12.5,///,3X,*RCENT*)

C BY ELEMENT, COMPUTE STRAIN & STRESS
DO 722 N=NSTART,NTOP,NSTEP
LRT=N+NCL
COUNT=1
DO 702 LL=N,LRT
SUMSIG=0.0
SUMEP=0.0
L=LL=NCL=1
DO 699 K=1,2
L=L+NCL+1
UFLM(1)=U(2*N(1(L))=1)
UFLM(2)=U(2*N(1(L))
UFLM(3)=U(2*NJ(L)=1)
UFLM(4)=U(2*NJ(L))
UFLM(5)=U(2*NM(L)=1)
UFLM(6)=U(2*NM(L))
SUMU=0.0
DO 500 I=1,6
500 SUMU=SUMU+UFLM(I)
IF(SUMU)501,490,501
490 DO 492 I=1,4
STRESS(I)=0.0
492 STRAIN(I)=0.0
GO TO 698
501 CALL BFORM(L,RBAR,A)
CALL JMPRD(B,UFLM,STRAIN,4,6,1)
CALL JMPRD(D,STRAIN,STRESS,4,4,1)
698 SUMSIG=SUMSIG+STRESS(1)
SUMEP=SUMEP+STRESS(3)
699 CONTINUE
   ICOUNT=ICOUNT+1
   ANS(1,ICOUNT)=.5*SUMSIG*.001
   ANS(2,ICOUNT)=.5*SUMEP*.001
702 CONTINUE
   ANS(1,1)=(R(NJ(L))+R(NM(L)))*.5
   WRITE(6,225)(ANS(1,J),J=1,NZZ)
   WRITE(6,224)(ANS(2,J),J=2,NZZ)
224 FORMAT(10X,23F5.2)
225 FORMAT(/,1X,F5.2,4X,24F5.2)
226 FORMAT(10X,24F5.1)
722 CONTINUE
   ANS(2,1)=DE*0.5+Z(NJ(N))
   DO 723 KZ=1,NZZ
   723 ANS(2,KZ+1)=ANS(2,KZ)+DE
      KZ=KZ+1
   WRITE(6,227)(ANS(2,J),J=1,KZ)
227 FORMAT(/,4X,'ZCENT=',24F5.2)
   WRITE(6,228)
230 FORMAT(/,20X,'EACH BLOCK IS R STRESS IN KSI',
     1/,36X,'Z STRESS IN KSI')
   WRITE(6,228)
228 FORMAT(1H1,/,57X,'NODAL DISPLACEMENTS',10X,'T=',E12.5,/,2X,
     1'RCENT')
   DO 726 LL=NSTART,NTOP,NSTEP
   ANS(1,1)=R(NI(LL))
   ANS(1,2)=U(2*NI(LL)=1)*1000000.
   ANS(2,2)=U(2*NI(LL)=1)*1000000.
   L=LL+1
   NTOPP=NZZ+1
DO 725 N=3, NTopp
L=L+1
ANS(1,N)=U(2*NM(L)=1)*1000000.
725 ANS(2,N)=U(2*NM(L))*1000000.
WRITE(6,229)(ANS(1,J), J=1,NTopp)
726 WRITE(6,226)(ANS(2,J), J=2,NTopp)
229 FORMAT(/,1X,F5.2,4X,24F5.1)
L=L+NCL+1
ANS(1,1)=R(NI(LL))
ANS(1,2)=U(2*NI(LL)=1)*1000000.
ANS(2,2)=U(2*NI(LL))*1000000.
L=LL=1
DO 727 N=3, NTopp
L=L+1
ANS(1,N)=U(2*NJ(L)=1)*1000000.
727 ANS(2,N)=U(2*NJ(L))*1000000.
WRITE(6,229)(ANS(1,J), J=1,NTopp)
WRITE(6,226)(ANS(2,J), J=2,NTopp)
ANS(2,1)=Z(NJ(Nstart))
DO 728 KZ=1, NZZ
728 ANS(2,KZ+1)=ANS(2,KZ)+DL
WRITE(6,227)(ANS(2,J), J=1, NZZ)
WRITE(6,231)
231 FORMAT(/,20X,'EACH BLOCK IS RADIAL DISPLACEMENT IN MICRON',
1" INCHES",/,'37X,' Z DISPLACEMENT IN MICROINCHES')
TPR=TPR+2.*DT
703 IF(T=TDT)271,460,460
703 T=T+DT
DO 350 N=1,NN
U(1)=0.0
UP1(1) = 0.0
IF(N=NFS)305,301,301
301 IFF = IFF + 1
    IF(IFF=NF)302,302,305
302 IF(Y=T)303,305,305
303 FORCE = F(IFF) * (1.0 + SIN(2.0PI*T/TTF=PI/2.)) * 0.5
    WFORCE = FORCE
    GO TO 306
305 FORCE = 0.0
306 IRR = 2*N - 1
    IRZ = IRR + 1
    SUMKR = 0.0
    SUMKUZ = 0.0
    DO 310 J = 1, 7
        IF(NADJ(N,J))349,349,307
307 IC = 2*J
    IUR = 2*NADJ(N,J) - 1
    IUZ = IUR + 1
    SUMKUR = SUMKUR + KASY(IRR, IC = 1) * U(IUR) + KASY(IRR, IC) * U(IUZ)
310 SUMKUZ = SUMKUZ + KASY(IRZ, IC = 1) * U(IUR) + KASY(IRZ, IC) * U(IUZ)
349 UP1(IRR) = 2*U(IRR) = UM1(IRR) = DT*DT*SUMKUR/MASS(IRR)
350 UP1(IRZ) = 2*U(IRZ) = UM1(IRZ) + DT*DT*(FORCE = SUMKUZ)/MASS(IRZ)
    GO TO 259
460 RETURN
END
SUBROUTINE JMPRD(A,B,RR,N,MM,L)
DIMENSION A(1),B(1),RR(1)
IR=1
DO 90 K=1,L
DO 90 J=1,N
RR(IR)=0.0
DO 80 I=1,MM
IA=N*(I-1)+J
IB=MM*(K-1)+I
80 RR(IR)=RR(IR)+A(IA)*B(IB)
90 IR=IR+1
RETURN
END
TIME INCREMENT, DT=0.83839E-07
TOTAL TIME OF PROPAGATION=0.66402E-05
ELEMENT SIZE=0.45207E-01
PLATE THICKNESS=9.2006E-02
YOUNG'S MODULUS=9.1177E-08
POISSON'S RATIO=9.31295E-01
WEIGHT DENSITY=9.10117E-02

DILATATION WAVE SPEED= 0.23912E 06INCHES PER SEC
DISTORTIONAL WAVE SPEED= 0.12486E 06INCHES PER SEC

D MATRIX

\[
\begin{bmatrix}
0.5875 & 0.3125 & 0.3125 & 0.0 \\
0.3125 & 0.6875 & 0.3125 & 0.0 \\
0.3125 & 0.6875 & 0.3125 & 0.0 \\
0.0 & 0.0 & 0.0 & 0.1875
\end{bmatrix}
\]

NF NELM NNOD NCL NFC NZ2 NZR
24 1564 840 22 33 24 46

FORCES

\[
\begin{bmatrix}
0.23122E 03 & 0.22117E 03 & 0.21112E 03 & 0.19776E 03 & 0.18996E 03 & 0.17900E 03 & 0.16385E 03 & 0.15982E 03 & 0.14746E 03 \\
0.13969E 03 & 0.12764E 03 & 0.11356E 03 & 0.10093E 03 & 0.93479E 02 & 0.82472E 02 & 0.70372E 02 & 0.60319E 02 & 0.50266E 02 & 0.43212E 02 \\
0.30159E 02 & 0.20156E 02 & 0.15063E 02 & 0.12566E 02
\end{bmatrix}
\]

NZRU
35 36 125 106 175 176 245 246 315 316 385 386 455 456 525 526 595 596 665 666 735 736 805 806 1 70 71 140 141 210 211 280 281 350 351 420 421 490 491 560 561 630 631 700 701 770 771 840

T= 0.0  FORCE= 0.127966E-01
T= 0.83639E-07  FORCE= 0.61150E-02
T= 0.16728E-06  FORCE= 0.24340E-01
T= 0.25092E-06  FORCE= 0.54321E-01