DYNAMICS OF COLUMN STABILITY WITH PARTIAL END RESTRAINTS

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ABSTRACT

The purpose of this investigation is to conduct a theoretical study of the dynamic behavior of columns with partial end restraints subjected to an axial dead load and a pulsating load. The governing differential equation is solved using a lumped impulse recurrence formula relative to time and coupled with a finite difference discretization along the member length. A computer program was developed to determine the first two critical frequencies as a function of end stiffness. Results obtained for a pinned ended column case compares very well with an exact analytical solution. Also, the natural frequency and buckling load equations are derived for equal and unequal end restraints.
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1. INTRODUCTION

1.1 Preliminary Remarks

A column is one of the most basic elements of a structural system. Although a number of studies have investigated the response of a column, the effects of partial end restraints, especially under dynamic loading, has not been well defined in the literature. There are three basic conditions of end restraints; 1) pinned, 2) fixed, and 3) partially restrained. The majority of analyses for a column considers the end restraint as either pinned or fixed; yet, most of the as built connections are neither pinned or fixed; they are partially restrained to some degree. Therefore, the concept of partial restraint is of a great practical interest. Another fairly undefined condition is that of a pulsating load on a column, such as the supports for an unbalanced rotating piece of machinery. Such a pulsating force under the right conditions can result in the instability of a column well below the Euler Buckling load. For pinned-end condition, the pulsating load case can be found in ref. 1, but no references to the case with partial end restraints exists. Therefore, the case of a column with partial end restraints and a pulsating load is of practical interest, and it is the subject of this investigation.

1.2 Literature Review

A literature review of the topic indicates a limited amount of research on the general idea of a pulsating load and no research
was found on the idea of partial end restraints with a pulsating load.

References 1 and 5 give the solution of a pinned-pinned column with a pulsating load $P_o + S_o \cos \Omega t$. The solution is in the form of a graph of the instability regions as a function of $\omega_o$, $\Omega$, $P_o$, $S_o$, and $P_e$. Reference 5 gives the instability boundary equations and a supporting table of values. A graph of the instability regions is shown on figure 3.

The case of a column with both ends fixed was discussed by F. Weidenhammer, Ingr.-Arch., vol. 19, page 162, 1951 (in German). Several stability problems under pulsating loads was found in the book by B. B. Bolotin, "Dynamic Stability of Elastic Systems," Moscow, 1956, (Russian).

1.3 Problem Statement

Figure 1 shows a schematic diagram of a column of length L that is bent about its weak axis with rotational end stiffnesses $K_1$ and $K_2$. The modulus of elasticity, E, and the moment of inertia, I, are constant through the length of the beam; and the axial load is represented by $P_o + S_o \cos \Omega t$.

The problem is to find the critical frequencies, $\Omega_i$, such that instability occurs before the buckling load is reached, and the critical frequencies should be in general terms of the parameters $K$, $E$, $I$, $L$, $\Omega$, and $\omega$. 

2
1.4 Objectives and Scope

The objectives of this study are;

1. Solve the differential equation using finite difference techniques.
2. Verify the solution against known benchmark cases.
3. Identify the first two critical frequencies in general terms.
4. Determine the natural frequencies and buckling load of a column with equal and unequal end restraints.

1.5 Assumptions

1. The column has a uniform cross section and is subjected to bending only about its' weak axis.
2. The material stress-strain relation is linear elastic.
3. The loads pass through the centroid of the beam resulting in no eccentric loading.
4. No local instability is considered.
5. The period of the pulsating force is very large in comparison with the longitudinal natural period of the column.
2. THEORETICAL DEVELOPMENT

2.1 Governing Equations

The governing differential equation of lateral vibrations for a column with damping and a pulsating load, \( P_o + S_o \cos \Omega t \), (see fig 1) is:

\[
EI \frac{\partial^4 w}{\partial x^4} + (P_o + S_o \cos \Omega t) \frac{\partial^2 w}{\partial x^2} + \rho \frac{\partial^2 w}{\partial t^2} + C \frac{\partial w}{\partial t} = 0
\]

The boundary conditions which include the effects of partial restraints are:

\[
EI \frac{\partial^2 w(0)}{\partial x^2} = K_1 \frac{\partial w(0)}{\partial x}
\]

\[
EI \frac{\partial^2 w(L)}{\partial x^2} = -K_2 \frac{\partial w(L)}{\partial x}
\]

\[
w(0) = w(L) = 0
\]
Equation (1) is associated with pure buckling. This implies that there is no lateral displacement prior to buckling. That would result in a highly complicated eigenvalue problem which is beyond the scope of this investigation. To circumvent this problem, an initial imperfection will be applied to the column. This will give a continuous displacement response of the column to the load. The exact solution would give discrete regions of instability; whereas with an initial imperfection, the response would asymptotically approach the theoretical regions of instability. Since all as built columns have some degree of imperfection, this assumption is valid. For the subject investigation, the initial imperfection will be of the form:

\[ \bar{w} = \delta \sin \frac{\pi x}{L} \]  

(5)

where \( \delta \) is the maximum initial displacement at the center of the column. This quantity remains variable so that the smaller \( \delta \) is made the closer the response gets to the exact solution. With initial imperfections, equation (1) becomes:

\[ EI \frac{d^4 w}{dx^4} + P(\frac{d^2 w}{dx^2} + \frac{d^2 \bar{w}}{dx^2}) + \rho \frac{d^2 w}{dt^2} + C \frac{\partial w}{\partial t} = 0 \]  

(6)
If equation (6) is rearranged such that known quantities are on the right hand side, we arrive at:

\[
EI \frac{\partial^4 w}{\partial x^4} + P \frac{\partial^2 w}{\partial x^2} + \rho \frac{\partial^2 w}{\partial t^2} + C \frac{\partial w}{\partial t} = -P \frac{\partial^2 w}{\partial x^2}
\]  

(7)

2.2 Non-Dimensional Equations

To generalize the results of this analysis, equation (7) will be put in non-dimensional form. To non-dimensionalize equation (7), the following variables are introduced.

\[
\tilde{\omega} = \frac{wL}{A}, \quad \tilde{\omega} = \frac{\tilde{w}L}{A}, \\
\tilde{x} = \frac{x}{L}, \quad \tilde{t} = \frac{t}{T_o}
\]

Substituting the above parameters into equation (7) and rearranging, we get;

\[
\frac{\partial^4 \tilde{w}}{\partial \tilde{x}^4} + \frac{P L^2}{EI} \frac{\partial^2 \tilde{w}}{\partial \tilde{x}^2} + \frac{\rho L^4}{EI T_o} \frac{\partial^2 \tilde{w}}{\partial \tilde{t}^2} + \frac{C L^4}{EI T_o} \frac{\partial \tilde{w}}{\partial \tilde{t}} = -\frac{P L^2}{EI} \frac{\partial^2 \tilde{w}}{\partial \tilde{x}^2}
\]  

(8)
To simplify equation (8), the following variables are defined:

\[ Z_1 = \frac{pL^2}{EI} = \left( P_0 + S_0 \cos \Omega t \right) \frac{L^2}{EI} \]

\[ Z_2 = \frac{pL^4}{EIT_o^2} \]

\[ Z_3 = \frac{C L^4}{EIT_o} \]

Therefore equation (8) becomes:

\[
\frac{\partial^4}{\partial x^4} \frac{\partial w}{\partial x} + Z_1 \frac{\partial^2}{\partial x^2} \frac{\partial^2 w}{\partial x^2} + Z_2 \frac{\partial^2}{\partial \xi^2} \frac{\partial^2 w}{\partial \xi^2} + Z_3 \frac{\partial w}{\partial \xi} = -Z_1 \frac{\partial^2 w}{\partial x^2} \tag{9} \]

The non-dimensional boundary conditions are:

\[ \dot{w}(0) = \dot{w}(1) = 0 \tag{10} \]

\[
\frac{\partial^2 \dot{w}(0)}{\partial x^2} = \frac{K_1 L}{EI} \frac{\partial \ddot{w}(0)}{\partial \xi} \tag{11} \]

\[
\frac{\partial^2 \dot{w}(1)}{\partial x^2} = -\frac{K_2 L}{EI} \frac{\partial \ddot{w}(1)}{\partial \xi} \tag{12} \]
2.3 Finite Difference Formulation

There are two basic steps in the solution of eq. (9):

1. First, a column of length L is divided into discrete sections of length h; and a central difference equation is written for each node to formulate the geometric stiffness matrix

\[
[K]\{w\} = \frac{\partial^4 w}{\partial x^4} + \frac{Z}{h^2} \frac{\partial^2 w}{\partial x^2} \tag{13}
\]

Where \{w\} is a vector of nodal displacements.

The central difference equation's used are (ref. 3);

\[
\frac{\partial w}{\partial x} = \frac{1}{2h} (w_{i-1} + w_{i+1}) \tag{14}
\]

\[
\frac{\partial^2 w}{\partial x^2} = \frac{1}{h^2} (w_{i-1} - 2w_i + w_{i+1}) \tag{15}
\]

\[
\frac{\partial^4 w}{\partial x^4} = \frac{1}{h^4} (w_{i-2} - 4w_{i-1} + 6w_i - 4w_{i+1} + w_{i+2}) \tag{16}
\]
2. Next, a lumped impulse recurrence formula is used to step the
response through a succession of discrete time intervals. The
equations used are (ref. 4);

\[ w^{(1)} = \frac{1}{2} \omega^{(0)} (\Delta t)^2 \] (17)

\[ w^{(s+1)} = 2w^{(s)} - w^{(s-1)} + \omega^{(s)} (\Delta t)^2 \] (18)

\[ \omega^{(s)} = \frac{w^{(s)} - w^{(s-1)}}{\Delta t} + \omega^{(s)} \frac{\Delta t}{2} \] (19)

In the above equations, the superscript indicates the time step.
The first equation gives the first time step and the second gives
each subsequent time step according to the two preceding steps.
From this procedure, the column's deflection vs. time can be
plotted. If the deflection constantly increases with time, then
the column is unstable.

2.3.1 Geometric Stiffness Matrix

The stiffness matrix equation (13), can be formulated for a
column of length L divided into 12 sections with 11 unknown
displacement nodes (fig. 2). Equation (13) can be written for all
eleven nodes. When writing the equation at nodes 10 and 11, the
fictitious nodes a and b must be expressed in terms of the other
nodes. To do this, the boundary conditions, equations (10) and (11) are written at the support nodes. The resulting matrix (II) is a function of the parameters $E$, $I$, $K_1$, $K_2$, $L$, $P_0$, $S_0$, $\Omega$, $\omega$, and $t$.

To solve for node $a$, equations (14) and (15) are substituted into equation (11) and rearranged to arrive at:

$$w_a = w_{11} Q_1$$

where

$$Q_1 = \frac{K_1 L \hat{h}}{2EI + 1}$$

For node $b$, equations (14) and (15) are substituted into equation (12), resulting in:

$$w_b = w_{10} Q_2$$

where

$$Q_2 = \frac{K_2 L \hat{h}}{2EI - 1}$$

Note that the non-dimensional element length, $\hat{h}$ is defined as:

$$\hat{h} = \frac{h}{L} = \frac{L}{12L} = \frac{1}{12}$$
Now equations (15) and (16) can substituted into equation (13) at all eleven nodes, resulting in the following nodal equations:

1) \[ \frac{\partial^4 w}{\partial x^4} = \frac{1}{h^4} (6\ddot{w}_1 - 4\ddot{w}_2 - 4\ddot{w}_3 + \ddot{w}_4 + \ddot{w}_5) \]

\[ Z \frac{\partial^2 \ddot{w}}{\partial x^2} = \frac{Z}{h^2} (-2\ddot{w}_1 + \ddot{w}_2 + \ddot{w}_3) \]

2) \[ \frac{\partial^4 w}{\partial x^4} = \frac{1}{h^4} (6\ddot{w}_2 - 4\ddot{w}_4 - 4\ddot{w}_1 + \ddot{w}_3 + \ddot{w}_6) \]

\[ Z \frac{\partial^2 \ddot{w}}{\partial x^2} = \frac{Z}{h^2} (-2\ddot{w}_2 + \ddot{w}_4 + \ddot{w}_1) \]

3) \[ \frac{\partial^4 w}{\partial x^4} = \frac{1}{h^4} (6\ddot{w}_3 - 4\ddot{w}_5 - 4\ddot{w}_1 + \ddot{w}_2 + \ddot{w}_7) \]

\[ Z \frac{\partial^2 \ddot{w}}{\partial x^2} = \frac{Z}{h^2} (-2\ddot{w}_3 + \ddot{w}_1 + \ddot{w}_5) \]
4) \( \frac{\partial^4 \dot{w}}{\partial x^4} = \frac{1}{h^4} (6\ddot{w}_4 - 4\ddot{w}_6 - 4\ddot{w}_2 + \ddot{w}_8 + \ddot{w}_1) \)

\[ Z_1 \frac{\partial^2 \dot{w}}{\partial x^2} = \frac{Z_1}{h^2} (-2\ddot{w}_4 + \ddot{w}_6 + \ddot{w}_2) \]

5) \( \frac{\partial^4 \dot{w}}{\partial x^4} = \frac{1}{h^4} (6\ddot{w}_5 - 4\ddot{w}_3 - 4\ddot{w}_7 + \ddot{w}_1 + \ddot{w}_9) \)

\[ Z_1 \frac{\partial^2 \dot{w}}{\partial x^2} = \frac{Z_1}{h^2} (-2\ddot{w}_5 + \ddot{w}_3 + \ddot{w}_7) \]

6) \( \frac{\partial^4 \dot{w}}{\partial x^4} = \frac{1}{h^4} (6\ddot{w}_6 - 4\ddot{w}_8 - 4\ddot{w}_4 + \ddot{w}_{10} + \ddot{w}_2) \)

\[ Z_1 \frac{\partial^2 \dot{w}}{\partial x^2} = \frac{Z_1}{h^2} (-2\ddot{w}_6 + \ddot{w}_8 + \ddot{w}_4) \]

7) \( \frac{\partial^4 \dot{w}}{\partial x^4} = \frac{1}{h^4} (6\ddot{w}_7 - 4\ddot{w}_5 - 4\ddot{w}_9 + \ddot{w}_3 + \ddot{w}_{11}) \)

\[ Z_1 \frac{\partial^2 \dot{w}}{\partial x^2} = \frac{Z_1}{h^2} (-2\ddot{w}_7 + \ddot{w}_5 + \ddot{w}_9) \]

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8) \( \frac{\partial^4 \ddot{w}}{\partial x^4} = \frac{1}{h^4} (6\ddot{w}_8 - 4\ddot{w}_{10} - 4\ddot{w}_6 + \ddot{w}_4) \)

\( z_1 \frac{\partial^2 \ddot{w}}{\partial \ddot{w}^2} = \frac{z_1}{h^2} (-2\ddot{w}_8 + \ddot{w}_{10} + \ddot{w}_6) \)

9) \( \frac{\partial^4 \ddot{w}}{\partial x^4} = \frac{1}{h^4} (6\ddot{w}_9 - 4\ddot{w}_7 - 4\ddot{w}_{11} + \ddot{w}_5) \)

\( z_1 \frac{\partial^2 \ddot{w}}{\partial x^2} = \frac{z_1}{h^2} (-2\ddot{w}_9 + \ddot{w}_7 + \ddot{w}_{11}) \)

10) \( \frac{\partial^4 \ddot{w}}{\partial x^4} = \frac{1}{h^4} (6\ddot{w}_{10} - 4\ddot{w}_8 + \ddot{w}_6 + \ddot{w}_{10} Q_2) \)

\( z_1 \frac{\partial^2 \ddot{w}}{\partial x^2} = \frac{z_1}{h^2} (-2\ddot{w}_{10} + \ddot{w}_8) \)

11) \( \frac{\partial^4 \ddot{w}}{\partial x^4} = \frac{1}{h^4} (6\ddot{w}_{11} - 4\ddot{w}_9 + \ddot{w}_7 + \ddot{w}_{11} Q_1) \)

\( z_1 \frac{\partial^2 \ddot{w}}{\partial x^2} = \frac{z_1}{h^2} (-2\ddot{w}_{11} + \ddot{w}_9) \)
The following variables are defined:

\[ \alpha = \frac{6}{h^4} \frac{2PL^2}{h^2EI} \]

\[ \beta = \frac{PL^2}{h^2EI} \frac{4}{h^2} \]

\[ \gamma = \frac{1}{h^4} \]

\[ \eta_1 = \frac{(6 + Q_1)}{h^4} \frac{2PL^2}{h^2EI} \]

\[ \eta_2 = \frac{(6 + Q_2)}{h^4} \frac{2PL^2}{h^2EI} \]

Using the above variables and the eleven nodal equations, the following stiffness matrix is formed.
2.3.2 Initial Imperfections

The initial imperfection, equation (5), and its second, derivative in non-dimensional form are;

\[
\ddot{w} - \frac{\delta L}{A} \sin \pi \dot{x}
\]

\[
\frac{\partial^2 \ddot{w}}{\partial x^2} = - \frac{\delta L \pi^2}{A} \sin \pi \dot{x}
\]

Now the vector \( \{ \ddot{w}_{xx} \} \) is defined as the non-dimensional initial imperfection curvatures;

\[
\begin{bmatrix}
1 \\
\sin 7\pi/12 \\
\sin 7\pi/12 \\
\sin 8\pi/12 \\
\sin 8\pi/12 \\
\sin 9\pi/12 \\
\sin 9\pi/12 \\
\sin 10\pi/12 \\
\sin 10\pi/12 \\
\sin 11\pi/12 \\
\sin 11\pi/12 
\end{bmatrix}
\]

2.3.3 Lumped Impulse Recurrence Formula

To apply the recurrence formulas equations (17), (18) and (19), the differential equation (9) is put into vector form. Using equations (20) and (21) and defining the following vectors,

\[
\{ \ddot{w} \} = \frac{\partial^2 \ddot{w}}{\partial t^2}
\]
\begin{equation}
\{\ddot{\omega}\} = \frac{\partial \ddot{\omega}}{\partial t}
\end{equation}

we arrived at the vector matrix form of equation (9) as;

\begin{equation}
[K] \{\ddot{\omega}\} + Z_2 \{\dddot{\omega}\} + Z_3 \{\dddot{\omega}\} = -Z_1 \{\dddot{w}_{xx}\}
\end{equation}

The non-dimensional time used in this formulation is;

\begin{equation}
\tilde{t} = \frac{t}{T_0}
\end{equation}

\Delta \tilde{t} = \frac{\tilde{t}_{i} - \tilde{t}_{i-1}}{T_0}

For the first time step, the acceleration vector is found from equation (21) to be;

\begin{equation}
\{\dddot{\omega}\}^{(s)} = -\frac{1}{Z_2} \left[ [K] \{\dddot{\omega}\}^{(s)} + Z_3 \{\dddot{\omega}\}^{(s)} + Z_1^{(s)} \{\dddot{w}_{xx}\} \right]
\end{equation}

If the above equation is substituted into equation (17), accounting for the condition that all terms are zero except the initial imperfection curvature at time \( t = 0 \), we get;
\[ \{w\}^{(1)} = -\frac{Z_1}{2Z_2} \left\{ \ddot{w}_{xx} \right\} (\Delta t)^2 \]  

(24)

Equation (24) is the first interval of the time sequence. For the rest of the intervals, we first substitute equation (19) into equation (23) and get:

\[
\{w\}^{(s)} = -\frac{1}{Z_2} \left[ \begin{bmatrix} K \end{bmatrix} \{\dot{w}\}^{(s)} + Z_1 \{\ddot{w}_{xx}\}^{(s)} + Z_2 \left[ \{\dot{w}\}^{(s)} - \frac{\{\dot{w}\}^{(s-1)}}{\Delta t} \right] - \frac{\{\ddot{w}\}^{(s)}}{2} \right]
\]

Then after solving the above equation explicitly for the acceleration vector and substituting it into equation (18), we get:

\[
\{\ddot{w}\}^{(s)} = \frac{\left[ \begin{bmatrix} K \end{bmatrix} \{\dot{w}\}^{(s)} + Z_1 \{\ddot{w}_{xx}\}^{(s)} + Z_2 \left[ \{\dot{w}\}^{(s)} - \frac{\{\dot{w}\}^{(s-1)}}{\Delta t} \right] \right]}{(Z_2 + \frac{Z_3 \Delta t}{2})}
\]

\[
\{\ddot{w}\}^{(s+1)} = 2\{\ddot{w}\}^{(s)} - \{\ddot{w}\}^{(s-1)} \cdot \left[ \begin{bmatrix} K \end{bmatrix} \{\dot{w}\}^{(s)} + Z_1 \{\ddot{w}_{xx}\}^{(s)} + Z_2 \left[ \{\dot{w}\}^{(s)} - \frac{\{\dot{w}\}^{(s-1)}}{\Delta t} \right] \right] \cdot \Delta t^2 \frac{(Z_2 + \frac{Z_3 \Delta t}{2})}{2}
\]

(25)

Equations (24) and (25) are the equations needed to step the response of the column through time.
3. RESULTS

3.1 Comparative Results

The case of a column with pinned ends and a pulsating force $P_o + S_o \cos \Omega t$ has been well documented, ref (1) and (5). The solution of the differential equation (1) with no damping takes the form;

$$ w = A f(t) \sin \frac{\Omega x}{L} $$

First, the above equation is substituted into the differential equation (1). Then the following variables are defined;

$$ \omega_0 = \frac{\pi^2 EI}{\rho L^2} $$

$$ \tau = \Omega t $$

$$ P_e = \frac{\pi^2 EI}{L^2} $$

$$ p = \frac{P_o}{P_e} $$

$$ s = \frac{S_o}{P_e} $$
Using the above variables, the transformed differential equation has the form:

\[
\frac{d^2f(r)}{dr^2} + (a + b \cos r) f(r) = 0
\]

where:

\[
a = \frac{\omega_0^2}{\Omega^2} (1-p)
\]

\[
b = -\frac{\omega_0^2}{\Omega^2} s
\]

The above differential equation is known as the Mathieu equation, and the character of the solution depends on the values of \(a\) and \(b\). For a particular set of values of \(a\) and \(b\), the solution is either stable or unstable. The unstable conditions are indicated by vibration which grow with time. Figure (3) is a plot of \(a\) vs \(b\) with the shaded region representing a stable condition and the unshaded areas indicating the regions of instability.

The computer program can now be compared to the pinned-pinned case as given by the Mathieu equation. The program was run using fixed values of \(a\) and \(\Omega\) with \(K_1=K_2=0\) for the pinned-pinned case. Then \(S_o\) was increased by increments of 1 lb until the response became unstable. Six cases were evaluated for the
condition of \( P_o = 0 \) and ten cases with \( P = 1/2 \, P_{cr} \). The variables used in the program are listed below:

\[
\begin{align*}
E &= 30 \times 10^6 \text{ psi} \\
I &= 0.0021 \text{ in}^4 \\
L &= 144 \text{ in} \\
A &= 0.0859 \text{ in}^2 \\
\rho &= 6.3 \times 10^{-5} \\
T_o &= 0.41749
\end{align*}
\]

The results are shown in table (1). The exact value of \( b \) corresponding to the value of \( b \) is from Figure (3) and a table of values from Reference (5). The comparison between the approximate solution and the exact solution is very good.

3.2 Critical Frequencies

The most valued result from this investigation would be to have an envelope similar to that of figure (3) as a function of \( K, L, E, I, \Omega, \omega, \rho, P_o, \) and \( S_o \). This would require the generation of hundreds of data points and a search for the correct relationship between the variables involved. An attempt was made to use an analogous axis of figure 3 with the exception that \( \omega_o \) became \( \omega \)

as a function of end stiffness and axial load. The results showed that there was not a direct correlation between the two, and that there was a shift along the \( \omega \) axis as the end stiffness increased. As a result, the idea of a total envelope was abandoned and only that of a more practical interest was pursued.
There are several practical considerations which would narrow the areas of practical interest. From Figure 3, there are stable regions such that \( P_o + S_o \geq P_{cr} \); however, any practical design code would tend to limit the maximum load \( P_o + S_o \) to be less than \( P_{cr} \). This will limit our concern to the lower half of the diagonal \( a - b \).

In addition the areas of critical interest are those areas in which very low values of \( S_o \) cause instability, and the first few critical frequencies cause such instability. Therefore, the first few frequencies will be determined for a range of end stiffnesses such that \( P_o + S_o \leq P_{cr} \).

The results of this analysis, to find the critical frequencies, do not include the effects of damping or unequal end restraints. In the program, the parameter \( C \) was set equal to zero and \( K_1 \) was always equal to \( K_2 \). To locate the critical frequencies the program was run repeatedly, starting with \( K=0 \) and its known critical frequencies and then \( K \) was increased and its subsequent frequencies were found.

The first two critical frequencies were easily determined for \( S_o \geq 5\% \ P_{cr} \). The remaining critical frequencies were more difficult to find; and they are not included in this report. Table 2 shows the first two critical frequencies for \( KL/EI \) varying from 0 to 2286 with \( P_o = 0 \). At \( KL/EI \) equal to 2000, \( P_{cr}/P_e \) is 3.992 which has a .2\% difference with that of the fixed-fixed case. Consequently with \( KL/EI = 2000 \), the ends are essentially fixed. It was found that as \( P_o \) increased, the relationship of \( \Omega/\omega \) remained constant when \( \omega \) is the natural frequency as a
function of end stiffness $K$ and axial load $P_0$. It was also found that there was a finite relationship between $KL/EI$ and $\Omega/\omega$. Figures 5 and 6 are graphs of $\Omega_1/\omega$ vs $KL/EI$ and $\Omega_2/\omega$ vs $KL/EI$, respectively. From these two graphs, the first two critical frequencies can be determined if $K$, $L$, $E$, $I$, and $\omega$ are known. The graphs start at $\Omega_1$ equal to $2\omega_0$ and $\Omega_2$ equal to $\omega_0$ with $K=0$ which is consistent with the pinned-pinned case. At $KL/EI = 2000$ $\Omega_1$ is equal to $1.91\omega$ and $\Omega_2$ is equal to $0.955\omega$. Changing the variables did little to effect the general behavior of the response at the critical frequencies. A typical response of $\Omega_1$ and $\Omega_2$ is on figures 7 and 8 respectively. The center plot on figures 7 and 8 is the critical frequency while the upper and lower plots show its stability at slightly above and below the critical frequency. Figure 7 shows a double beat corresponding to the critical frequency at about twice the natural frequency; and at the critical frequency, the double beat fades as it becomes unstable.

Appendix A formulates the natural frequency and buckling load for a column with equal and unequal end restraints. Figures 9 and 10 are graphs of the dimensionless natural period vs $KL/EI$ for a range of axial loads. From the graph, either the natural period or the stiffness can be determined if the other is known. With the natural period and the end stiffness known, the first two critical frequencies can be determined from figures 5 and 6. In addition $P_{cr}/P_e$ vs $KL/EI$ is graphed on figures 11 and 12 with specific values of the graph in table 3. Therefore, if the end stiffness or the natural frequency can be estimated or determined
experimentally, the first two critical frequencies can be found by using figures 5, 6, 9 and 10, and the static buckling load can be determined from figures 11 and 12 or table 3.
4. CONCLUSIONS AND FUTURE RESEARCH

4.1 Conclusions

The following conclusions can be made from this report:

1. A numerical analysis program is written (Appendix B) such that all input quantities are variable and the response of deflection vs time can be plotted.

2. The program gives results which compare very well for the pinned-pinned case (table 1).

3. The natural frequencies and buckling load formulas of a column with equal and unequal end restraints have been developed (Appendix A).

4. The dimensionless natural period vs KL/EI has been plotted for a range of axial loads (figures 9 and 10).

5. The first two critical frequencies have been determined (figures 5 and 6) as a function of K, L, E, I, and $\omega$.

6. If either the end stiffness $K$ or the natural period $T_0$ is known, the other can be evaluated from figures 9 and 10. With the end stiffness and natural period known, the first two critical frequencies can be found from figures 5 and 6.

4.2 Future Research

Although the solution procedure presented in this report did include damping and unequal end restraints, no studies have been conducted concerning the effects of these two variables. Therefore, future research can be performed to investigate the effect of damping and unequal end restraints on the critical frequencies. Future work may also include determining all the
critical frequencies as only two have been determined in this report. Also, the effects of material plasticity on the critical frequencies could be the subject of future investigations. In addition, testing could be conducted to verify the results herein.
NOMENCLATURE

A area
C damping coefficient
E modulus of elasticity
f natural frequency
h finite difference element length
h dimensionless finite difference element length
I moment of inertia
K geometric stiffness matrix
K₁ rotational stiffness at end 1
K₂ rotational stiffness at end 2
L length of column
P P₀ + S₀ cos Ωt
Pₚₗ critical buckling load
Pₑ Euler buckling load
P₀ dead load
S₀ pulsating load
T₀ natural period
t time
\( \bar{t} \) dimensionless time
w deflection
\( \bar{w} \) initial imperfection displacement
\( \ddot{w} \) dimensionless deflection
\( \dddot{w} \) dimensionless initial imperfection
\( \dot{w} \) velocity
\( \dot{\bar{w}} \) dimensionless velocity
\( w_c \) centerline deflection excluding initial imperfection
\( \ddot{w} \)  acceleration
\( \ddot{w} \)  dimensionless acceleration
\( X \)  axial distance
\( \overline{X} \)  dimensionless axial distance
\( \rho \)  density per unit length
\( \delta \)  maximum centerline initial imperfection
\( \omega \)  natural frequency with effects of end stiffness \( K \) and axial load \( P_0 \)
\( \Omega_1 \)  first critical frequency
\( \Omega_2 \)  second critical frequency
\( \Omega \)  forcing function
REFERENCES


Table 1. Comparison of Exact vs Computer Approximation for a Pinned-Pinned Column

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Variables used:  
- $E = 30 \times 10^6$ psi  
- $I = .0021$ in$^4$  
- $L = 144$ in  
- $L/r = 920$
Table 3. KL/EI vs $P_{cr}/P_e$ for Equal End Stiffness

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FIGURES
Figure 1. Problem Definition
Figure 2. Finite Difference Node Definition
Figure 3. Plot of Instability Regions for a Pinned-Pinned Column (Ref. 1 and 5)
Figure 4. Effect on the Time Increment ($\Delta t$) on the Numerical Stability of the Program

- $\Delta t = 0.0020$ Stable
- $\Delta t = 0.0033$ Stable
- $\Delta t = 0.0034$ Unstable
Figure 5. Plot of $\Omega_1/\omega$ vs KL/EI for Equal End Stiffness
Figure 6. Plot of $\Omega_2/\omega$ vs KL/EI for Equal End Stiffness
Figure 7. Typical Deflection - Time Response at the First Critical Frequency (K=2000, S=4)
Figure 8. Typical Deflection - Time Response at the Second Critical Frequency (K=2000, S=4)

\[ \Omega = \omega \]

(Critical Frequency)

\[ \Omega = 0.97\omega \]

\[ \Omega = 0.94\omega \]
Figure 9. Graph of Equation (h) for Certain Values of $P/P_e$ with $KL/EI$ Varying From 0 to 1000. $P/P_e$ Varies From 0 to 3.5 in .5 Increments With the Final One Equal to 3.75.
Figure 10. Graph of Equation (h) with the same $P/P_e$ increments as Figure 9, but with a smaller range for $KL/EI$. 
Figure 11. Plot of $P_{cr}/P_e$ vs KL/EI for Equal End Stiffness
With KL/EI Varying From 0 to 2000
Figure 12. Plot of $P_{cr}/P_e$ vs KL/EI for Equal End Stiffness
With KL/EI Varying From 0 to 200
APPENDIX A
APPENDIX A

NATURAL FREQUENCY AND BUCKLING LOAD

A.1 Equal End Stiffnesses

To obtain the natural frequency consider equation (1) with no pulsating force, \( S_0 = 0 \). Then the differential equation (1) becomes:

\[
\text{EI} \frac{\partial^4 w}{\partial x^4} + p_o \frac{\partial^2 w}{\partial x^2} + \rho \frac{\partial^2 w}{\partial t^2} + C \frac{\partial w}{\partial t} = 0
\] (a)

This partial differential equation can be separated into two ordinary differential equations by separation of variables. Letting \( w(x,t) = \tilde{W}(x)T(t) \) we get:

\[
\frac{d^4 \tilde{W}}{dx^4} + k^2 \frac{d^2 \tilde{W}}{dx^2} - \lambda \tilde{W} = 0
\] (b)

\[
\frac{d^2 T}{dt^2} + \frac{C}{\rho} \frac{dT}{dt} + \omega^2 T = 0
\] (c)
where  \[ \omega = \sqrt{\frac{EI\lambda}{\rho}} \]  (natural circular frequency)  \( (d) \)

\[ k = \sqrt{\frac{P_0}{EI}} \]  \( (e) \)

The exact solution to equation (b) leads to a transcendental equation which cannot be solved explicitly. Approximate solutions have been formulated and one in particular will be discussed here (ref. 2). In the solution of (b), from ref. 2, a shape function was chosen to satisfy the boundary conditions then the error was minimized by invoking the Galerkin criterion to solve for \( \lambda \).

The shape function used was:

\[ W(x) = A \left[ \sin \frac{\pi x}{L} + B(1 - \cos \frac{2\pi x}{L}) \right] \]  \( (f) \)

where  \( B \) can be determined from the boundary conditions, equation (2) and (3):

\[ B = \frac{KL}{4\pi EI} \]  \( (g) \)
After invoking Galerkins criterions an explicit expression for \( \lambda \) is obtained. By using equation (d) to get the undamped circular frequency and noting that the natural frequency is;

\[
f = \frac{\omega}{2\pi} \text{ (cps)}
\]

we get the natural frequency of a column with equal end restraints to be;

\[
f = \frac{1}{2L} \sqrt{\frac{\text{EI}}{\rho}} \sqrt{\frac{12(\pi\text{EI})^2 \left( \frac{\pi^2}{L^2} - \frac{P}{\text{EI}} \right) + 32 \text{EIKL} \left( \frac{5\pi^2}{2L^2} - \frac{P}{\text{EI}} \right) + 3(\text{KL})^2 \left( \frac{4\pi^2}{L^2} - \frac{P}{\text{EI}} \right)}{(12(\pi\text{EI})^2 + 32 \text{EIKL} + \frac{9}{4} (\text{KL})^2)}}
\]

As \( P \) goes to the buckling load \( P_{cr} \), the natural frequency goes to zero. Therefore by setting \( f = 0 \) we get the buckling load to be;

\[
P_{cr} = \frac{\pi^2 \text{EI}}{L^2} \left[ \frac{12(\pi\text{EI})^2 + 80 \text{EIKL} + 12(\text{KL})^2}{12(\pi\text{EI})^2 + 32 \text{EIKL} + 3(\text{KL})^2} \right]
\]
The frequency and buckling equations above can be put into a useful graph form by setting the independent variable to be \( KL/EI \). By doing this the following equations are formed;

\[
\frac{T_0}{L^2} \sqrt{\frac{EI}{\rho}} = -\pi \sqrt{\frac{12\pi^2}{12\pi^2 + 32 \left(\frac{KL}{EI}\right) + 9 \frac{KL^2}{EI}} + \frac{9}{4} \frac{KL^2}{EI} + \frac{KL^2}{EI}} \]

\[
\frac{P_{cr}}{P_e} = \frac{12\pi^2 + 80 \frac{KL}{EI} + 12 \frac{KL^2}{EI}}{12\pi^2 + 32 \frac{KL}{EI} + 3 \frac{KL^2}{EI}}
\]

For equation (i), when \( K = 0 \) we get the buckling load to be;

\[
\frac{P_{cr}}{P_e} = \frac{\pi^2 EI}{L^2}
\]

which is correct for a pinned-pinned column as \( K = \infty \) we get the buckling load to be;

\[
\frac{P_{cr}}{P_e} = 4 \frac{\pi^2 EI}{L^2}
\]

which is correct for a fixed-fixed column. Since the upper and
lower bounds of the equation are correct, and since the Galerkin method minimizes the error continuously through the domain, then equation (J) is a very good approximation to the buckling load of a column with equal end restraints. Equation h is graphed on figures 9 and 10, and equation i is graphed on figure 11.

A.2 Unequal End Stiffnesses

To obtain the natural frequency and buckling load of a column with unequal end stiffnesses a polynominal shape function was used with the Galerkin criterion. The differential equation (b) was solve using the following shape function;

\[ W = a_0 + a_1X + a_2X^2 + a_3X^3 + a_4X^4 \]

To satisfy the boundary conditions the above shape function becomes;

\[ W = a_1 \left[ X + \frac{(K_1K_2L^4 + 6EI K_1L^3)}{(2EI K_2L^4 + 12(EI)^2L^3)} X^2 \right. \]

\[ \left. - \frac{(2K_1K_2L^3 + 6EI K_2L^2 + 10EI K_1L^2 + 24(EI)^2L)}{(2EI K_2L^4 + 12(EI)^2L^3)} X^3 \right. \]

\[ \left. + \frac{(K_1K_2L^2 + 4EI K_2L + 4EI K_1L + 12(EI)^2)}{(2EI K_2L^4 + 12(EI)^2L^3)} X^4 \right] \]
After invoking Galerkins criterion, the natural frequency of a column with unequal end stiffnesses is obtained.

\[
f = \frac{1}{2\pi} \sqrt{\frac{EI}{\rho}} \left( \frac{C_4 L^4 + C_3 L^3 + \ldots + C_0}{D_8 L^8 + D_7 L^7 + \ldots + D_4 L^4} \right) - P \left( B_6 L^6 + B_5 L^5 + \ldots + B_2 L^2 \right)
\]

where;

\[
C_0 = 435456 \, (EI)^5
\]
\[
C_1 = 199584 \, (EI)^4 \, (K_1 + K_2)
\]
\[
C_2 = 18144 \, (EI)^3 \, [K_1^2 + 13K_1K_2/3 + K_2^2]
\]
\[
C_3 = 6552 \, (EI)^2 \, [K_1K_2^2 + K_1^2K_2]
\]
\[
C_4 = 504 \, EI \, K_1^2K_2^2
\]
\[
B_2 = 44064 \, (EI)^4
\]
\[
B_3 = 11232 \, (EI)^3 \, (K_1 + K_2)
\]
\[
B_4 = 864 \, (EI)^2 \, [K_1^2 + 35K_1K_2/12 + K_2^2]
\]
\[
B_5 = 180 \, EI \, (K_1K_2^2 + K_1^2K_2)
\]
\[
B_6 = 12 \, K_1^2K_2^2
\]
\[
D_4 = 4464 \, (EI)^5
\]
\[
D_5 = 1140 \, (EI)^4 \, (K_1 + K_2)
\]
\[
D_6 = 76 \, (EI)^3 \, [K_1^2 + 68K_1K_2/19 + K_2^2]
\]
\[
D_7 = 17 \, (EI)^2 \, (K_1K_2^2 + K_1^2K_2)
\]
\[
D_8 = EI \, K_1^2K_2^2
\]

As \( P \) goes to the buckling load \( P_{cr} \), the natural frequency goes to
zero. Therefore by setting $f = 0$ we get the buckling load to be:

$$P_{cr} = \frac{(C_4L^4 + C_3L^3 + \ldots + C_0)}{(B_6L^6 + B_5L^5 + \ldots + B_2L^2)}$$
APPENDIX B
APPENDIX B

THE COMPUTER PROGRAM

The program was written in Fortran 5 and the program listing is on page , Appendix B. There are seven main components of the program.

1. **Constants;** This is where the constants in the program are set up including the columns properties and several dummy variables that are used in the program.

2. **Initial Imperfections;** This is where the initial imperfections are set up using equation 21.

3. **First and Second Data Points;** This portion defines the first two data points. The first being at time zero and the second at the first time step using equation 24.

4. **Remaining Data Points;** This section defines the remaining data points using equation 25.

During each time increment the deflection of all eleven nodes are calculated but only the center node (node 1) is saved for all time steps. This reduces the amount of array storage needed for each run which intern enables an increased number
of time steps to be saved per run due to computer storage limitations.

5. **Check Data**; This routine checks the data to determine if the response is stable or unstable. Essentially the routine checks each peak and if two successive peaks are less than the preceding peak then the response is stable. Due to the wide range of possible responses, this routine is not 100% reliable. Therefore this routine is only an indication of the response and should not be used as a definite check on the stability of the response.

6. **Plot Results**; This routine plots the dimensionless displacement vs dimensionless time. This routine is particular to the computer being used and it therefore cannot be used on any other computer that does not have the plotting subroutines available.

7. **Stiffness Matrix Subroutine**; This subroutine evaluates the stiffness matrix as described by equation (20).

One critical considerations in this analysis is the time step to be used. The program was tested with constants from Section 3.1 and $P/P_{cr} = .5$, $S/P_{cr} = .5$, $K = 2000$, $\Omega = 20$ and the time step was varied between .001 to .0035. Figure 4 shows the maximum mid-span deflection, $w_c$ (in.) vs time, $t$ (sec.) and it shows that the maximum time step that can be used is .0033 sec. At $\Delta T = .0034$ sec. and greater the analysis becomes numerically unstable. The
critical time step was particular to the variables used and it is recommended that the time interval be no larger than one percent of the natural period of the column. The initial imperfection was chosen to be two orders of magnitude smaller than the tolerance of a typical column.
COMPUTER PROGRAM

FINITE DIFFERENCE ROUTINE TO DETERMINE THE
DYNAMIC RESPONSE OF A COLUMN TO A DEAD LOAD
AND A PULSATING LOAD WITH UNEQUAL END RESTRAINTS

PROGRAM COLBUCK

DIMENSION WI(I1),K(I1,11),WBAR(I1),XD(5000),YD(5000)
DIMENSION ZZ1(I1),ZZ2(I1),ZZ3(I1),ZZ5(I1),WIM1(I1),WIM2(I1)
REAL KK1,KK2,L,IX,K,H
CHARACTER*20 XCHAR
CHARACTER*20 YCHAR

PRINT *, 'INPUT OMEGA,S'
READ *, OMEGA,S

************************** CONSTANTS **************************

DT=.0020
PI=3.1415927
DELTA=.00001
E=30E+6
IX=.0021
L=144.
H=1/12.
KK1=87500.0
KK2=87500.0
AA=.0859
P=58.8
TO=.260413
RHO=6.3E-5
CC0=0.0
DTB=DT/TO
EI=E*I X
Z2=RHO*L**4/(EI*TO**2)
Z3=CC0*L**4/(EI*TO)
Q1=(KK1*L*H/(2.0*EI)-1)/(KK1*L*H/(2.0*EI)+1)
Q2=(KK2*L*H/(2.0*EI)-1)/(KK2*L*H/(2.0*EI)+1)
IFLAG=0

*************** INITIAL IMPERFECTIONS ***************

BI=0.0
WBAR(1)=-(PI**2)*L*DELTA/AA
DO 10 I=2,10,2
BI=BI+1
WBAR(I)=WBAR(I)*SIN(PI*(6+BI)/12)
10 WBAR(I+1)=WBAR(I)

99
FIRST & SECOND DATA POINTS

\[
\text{DO 20 I}=1,11
\]
\[
\text{WIM1}(I)=0.0
\]
\[
\text{W1}(I)=((F'+S) \times L \times 2 \times \text{WBAR}(I) \times \text{DTB}^2)/(2 \times Z2 \times EI)
\]
\[
\text{YD}(1)=\text{WIM1}(1)
\]
\[
\text{YD}(2)=\text{W1}(1)
\]
\[
\text{XD}(1)=0.0
\]
\[
\text{XD}(2)=\text{DT}
\]

REMAINING DATA POINTS

\[
\text{DO 30 I}=3,5000
\]
\[
\text{DO 35 J}=1,11
\]
\[
\text{WIM2}(J)=\text{WIM1}(J)
\]
\[
\text{WIM1}(J)=\text{W1}(J)
\]
\[
\text{XD}(I)=\text{DT} \times (I-1)
\]
\[
\text{T}=\text{DT} \times (I-2)
\]
\[
\text{ZI}=(F+S+COS(OMEGA*T)) \times L \times 2/EI
\]
\[
\text{CALL XKM(ZI,H,Q1,Q2,K)}
\]
\[
\text{DO 40 II}=1,11
\]
\[
\text{SUM}=0
\]
\[
\text{DO 50 JJ}=1,11
\]
\[
\text{SUM}=\text{SUM}+K(II,JJ) \times \text{WIM1}(JJ)
\]
\[
\text{ZZ1}(J)=2 \times \text{WIM1}(J)-\text{WIM2}(J)
\]
\[
\text{ZZ2}(J)=\text{Z1} \times \text{WEAR}(J)+\text{ZZ3}(J)+((Z3/\text{DTB}) \times (\text{WIM1}(J)-\text{WIM2}(J)))
\]
\[
\text{ZZS}(J)=((\text{ZZ2}(J) \times \text{DTB}^2)/(Z2+Z_*\text{DTB}/2))
\]
\[
\text{WI}(J)=\text{ZZ1}(J)-\text{ZZS}(J)
\]
\[
\text{YD}(I)=\text{WI}(I)
\]
\[
\text{CONTINUE}
\]

CHECK DATA

\[
\text{YDFKNEW}=0.0
\]
\[
\text{ICOUNT}=0
\]
\[
\text{DO 55 I}=2,4999
\]
\[
\text{YB}=\text{YD}(I-1)
\]
\[
\text{YN}=\text{YD}(I)
\]
\[
\text{YA}=\text{YD}(I+1)
\]
\[
\text{IF} (\text{YB}.LT.\text{YN}.AND.\text{YA}.LT.\text{YN}) \text{THEN}
\]
\[
\text{YDFKOLD}=\text{YDFKNEW}
\]
\[
\text{YDFKNEW}=\text{YN}
\]
\[
\text{IF} (\text{YDFKNEW}.GT.\text{YDFKOLD}) \text{ICOUNT}=0
\]
\[
\text{IF} (\text{YDFKNEW}.LT.\text{YDFKOLD}) \text{ICOUNT}=\text{ICOUNT}+1
\]
\[
\text{IF} (\text{ICOUNT.EQ.2}) \text{GOTO 65}
\]
\[
\text{END IF}
\]
\[
\text{CONTINUE}
\]
\[
\text{PRINT 
"*** UNSTABLE ***"}
\]
\[
\text{IFLAG}=2
\]
\[
\text{GOTO 75}
\]
\[
\text{PRINT 
"*** STABLE ***"}
\]
IFLAG=1
75 CONTINUE
IF(IFLAG.EQ.2)GOTO 200
200 CONTINUE
*
PRINT *, ' TYPE'
PRINT *, ' 1) TO PLOT'
PRINT *, ' 2) TO TRY AGAIN'
READ *, IFLAG
IF(IFLAG.EQ.2)GOTO 5
*
**************** PLOT RESULTS ********************
*
XCHAR='T/TO
YCHAR='WL/A
CALL PSEUDO
CALL INFOPLT(1,5000,XD,1,YD,1.0,
*0.0,1.0,0.0,13,XCHAR,16,YCHAR,0)
CALL CALFLT(0,0,99)
*
STOP
END
*
******************** STIFFNESS MATRIX SUBROUTINE ****************
*
SUBROUTINE XKM(Z1,H,Q1,Q2,K)
DIMENSION K(11,11)
REAL K,H,H2,H4
H2=1.0/H**2
H4=1.0/H**4
A=6*H4-2*Z1*H2
B=Z1*H2-4*H4
C=H4
D1=H4*(6+Q1)-2*Z1*H2
D2=H4*(6+Q2)-2*Z1*H2
K(1,1)=A
K(1,2)=B
K(1,3)=B
K(1,4)=C
K(1,5)=C
K(1,6)=0.0
K(1,7)=0.0
K(1,8)=0.0
K(1,9)=0.0
K(1,10)=0.0
K(1,11)=0.0
K(2,2)=A
K(2,3)=C
K(2,4)=B
K(2,5)=0.0
K(2,6)=C
K(2,7)=0.0
K(2,8)=0.0
K(2,9)=0.0

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\begin{align*}
K(2,10) &= 0.0 \\
K(2,11) &= 0.0 \\
K(3,3) &= A \\
K(3,4) &= 0.0 \\
K(3,5) &= B \\
K(3,6) &= 0.0 \\
K(3,7) &= C \\
K(3,8) &= 0.0 \\
K(3,9) &= 0.0 \\
K(3,10) &= 0.0 \\
K(3,11) &= 0.0 \\
K(4,4) &= A \\
K(4,5) &= 0.0 \\
K(4,6) &= B \\
K(4,7) &= 0.0 \\
K(4,8) &= C \\
K(4,9) &= 0.0 \\
K(4,10) &= 0.0 \\
K(5,5) &= A \\
K(5,6) &= 0.0 \\
K(5,7) &= B \\
K(5,8) &= 0.0 \\
K(5,9) &= C \\
K(5,10) &= 0.0 \\
K(5,11) &= 0.0 \\
K(6,6) &= A \\
K(6,7) &= 0.0 \\
K(6,8) &= B \\
K(6,9) &= 0.0 \\
K(6,10) &= C \\
K(6,11) &= 0.0 \\
K(7,7) &= A \\
K(7,8) &= 0.0 \\
K(7,9) &= B \\
K(7,10) &= 0.0 \\
K(7,11) &= C \\
K(8,8) &= A \\
K(8,9) &= 0.0 \\
K(8,10) &= B \\
K(8,11) &= 0.0 \\
K(9,9) &= A \\
K(9,10) &= 0.0 \\
K(9,11) &= B \\
K(10,10) &= D2 \\
K(10,11) &= 0.0 \\
K(11,11) &= D1 \\
\text{DO} 15 \text{ I}=1,11 \\
\text{DO} 15 \text{ J}=1,11 \\
K(J,I) &:= (I,J) \\
\text{RETURN} \\
\text{END}
\end{align*}
The aim of this investigation is to conduct a theoretical study of the dynamic behavior of columns with partial end restraints and loads consisting of a dead load and a pulsating load. The differential equation is solved using a lumped impulse recurrence formula relative to time coupled with a finite difference discretization along the member length. A computer program is written from which the first critical frequencies are found as a function of end stiffness. The case of a pinned ended column compares very well with the exact solution. Also, the natural frequency and buckling load formulas are derived for equal and unequal end restraints.