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**DYNAMIC LOAD MODEL STUDY FOR
OVERHEAD TRANSMISSION LINES**

**FINAL REPORT FOR THE
PERIOD NOV. 1975 TO SEP. 1976**

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DYNAMIC LOAD MODEL STUDY FOR
OVERHEAD TRANSMISSION
LINES

ERDA Contract No. E(49-18)-2063

Report By: Research Consulting Associates
Lexington, Massachusetts

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EXECUTIVE SUMMARY

The research project described in this document was initiated and supported by the Electric Energy Systems Division, Office of Conservation, U.S. Energy Research and Development Administration. Its primary objective is to examine by scale models the dynamic longitudinal loadings on flexible transmission structures, the emphasis being on the dynamic aspects of the problem.

In the course of the investigation certain new concepts have also been developed concerning the elements of a transmission line system which contributes to the local flexibility of individual structures. Further, new methods of analysis have been devised for the future use of designers in this field.

These model test results have demonstrated that it is possible to simulate a number of dynamic loading conditions on a scale model of an actual transmission line system, taking into account a large number of variables, and producing results having direct interpretation at full scale.

The method appears to be particularly useful in identifying any potential "trouble areas" in an existing design such as the use of horizontal struts on light angle structures, and/or adequacy of fatigue strength under galloping conditions.

A sample of the principal conclusions of this investigation are:

- 1) A method for rapidly calculating the flexibility (stiffness) of steel pole transmission structures has been developed. The method

relies on computer print-out data supplied by pole manufactures at the time of bid evaluation. 2) Results of broken static wire tests show that structure flexibility reduces both the dynamic load and the residual load below the load calculated for a rigid structure. 3) The maximum loads occur for broken static wire on the most flexible structure, and for ice unload of static wire on the most stiff structure. The latter result is subject to the further restraint of a horizontal strut at a light angle. 4) Unexpectedly high loads were found on the stiffer structure with long insulators (including the horizontal strut) for galloping conductor and galloping static wire tests.

A somewhat surprising result is the high dynamic loading of the stiffer structure under galloping test. The effect of the higher stiffness is pronounced, being almost twice the loading under galloping conductor on the stiffer structure for short insulators.

The most startling comparison however, was for long insulators. Here, the combined effect of higher stiffness and horizontal strut restraint leads to galloping loads which are between 4 and 5 times higher on the stiffer structure. Further, the galloping static wire loads are about $\frac{2}{3}$ of the static wire initial tension. This is considered to be a significant finding, and may bring serious concern to designers. The primary concern being the possible fatigue damage to cross arms and cross arm supports.

1. A BRIEF INTRODUCTION TO THE PROBLEM

The specifications and design of transmission structures to withstand longitudinal loading has been a subject for discussion, debate, and philosophical opinion for many years. There is, at the present time, no widely accepted design specification, or loading criteria, although an inspection of a "typical utility" specification for transmission structure loadings would find one or more of the following loading conditions spelled out:

- 1) Broken Conductor
- 2) Broken Shield Wire
- 3) Unbalanced Ice
- 4) Stringing Load

The extent to which any of the above loadings is "weighted" relative to the traditional transverse loadings often is based on individual utility experience. A fifth condition--galloping loadings--has also been included by at least one utility because of some recent galloping experience.

The designer applies the various loading specifications to develop a critical loading distribution for a particular structure, and as an example, may find that the lower half of the structure must be designed to withstand the transverse loads, while the upper half of the structure must be designed to withstand the longitudinal loads. The delivered structure, together with extensive computer analysis, thus is represented as having met all specified critical loading requirements. A static test often verifies the analysis.

This approach may fall short of a rational design procedure in several respects:

- 1) The structure is assumed to be isolated from the other structures in the system.
- 2) The loads are applied statically in the design process.
- 3) The effects of flexibility are neglected except for the additional moments due to vertical loads on a deflected structure.

While it is now well-known that longitudinal flexibility reduces the bending moments below a broken conductor due to additional support provided by the intact shield wire, there is not sufficient information yet available to apply this beneficial effect in rational design. The missing link is the dynamic reaction of the flexible structure following the break, and which intuitively must generate higher loads.

Unfortunately, the analytical prediction of dynamic loading effects in a system of flexible structures is not as well developed as the static loading analysis of isolated structures. This is especially so in the case of real systems where non-uniform spans, angle structures, non-uniform height, and variable stiffness are the rule, not the exception. Analysis, if attempted, is also complicated by non-linear elements such as the effects of conductor tension unloading in suspension insulators.

Notwithstanding such practical difficulties, the goal of achieving a more rational design procedure should be pursued and can easily be justified. For example, if present structures are "underdesigned" and fail in service, the cost of even the most expensive rational design analysis is certain to be a small fraction of the cost of the failure. On the other

hand, if present structures are being over-designed, again the cost of the most expensive rational design analysis can easily be recovered in reduced structure and foundation costs. Either way, the incremental costs of more rational design are justified.

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In the course of the investigation certain new concepts have also been developed concerning the elements of a transmission line system which contributes to the local flexibility of individual structures. Further, new methods of analysis have been devised for the future use of designers in this field.

2. "FULL-SCALE TRANSMISSION LINE"

DESCRIPTION AND ANALYSIS

A full-scale transmission line, constructed early in 1976 was selected for model design analysis. The transmission structures are steel poles, supporting a 115 kV single circuit over the total line length, and a 23 kV underbuilt section over a portion of its length. A plan view of the line is shown in Figure 1. The line is located in Billerica, Massachusetts and is owned by the New England Electric Company, who supplied all of the required detail design data.

A section of this line, consisting of eleven spans between deadend structures, was selected for model design and testing. There are a total of three light angle structures and one heavy angle structure (45 degrees). The remaining structures are tangent. Table 1 lists some basic design information. The span lengths range as follows from structure Number 22.

Structure Number	23	24	25	26	27	28	29	30	31	32	33
Back Span (Feet)	478	478	478	366	380	374	369	360	415	417	262

The underbuilt section begins at structure 29 and ends at structure 33.

Conductor, shield-wire specifications are:

CONDUCTOR: 795,000CM ACSR Condor
115 kV and 23kV
5000 lbs. Tension @ $\frac{1}{2}$ " ice 4 psf wind

SHIELD WIRE: 7 Strand/No. 9 Alumoweld
3000 lbs. Tension @ $\frac{1}{2}$ " ice 4 psf wind

The estimated weights per foot under these conditions are 2.2 lb/ft and 1.1 lb.ft

FIGURE 1.
NEW ENGLAND POWER COMPANY MAP SHOWING
PROPOSED ELECTRIC POWER EXTENDING FROM
A TAP POINT WITH M-139 LINE TO THE
BILLERICA SUBSTATION IN BILLERICA, MASS.

TABLE 1
REFERENCE DATA FOR THE BILLERICA FULL-SCALE LINE

Structure I.D. No.	Angle Ref. (deg.)	Hgt. (ft.)	Base Dia. (in.)	Top Dia. (in.)	Design GLM (in-Kips)	Base Thickness (in.)
BT-82 DE: 22	DE	80	xxx	xxx	xxx	xxx
BT-3 : 23	0	85	20.7	10.0	5,250	0.2500
BT-3 : 24	0	85	20.7	10.0	5,250	0.2500
BT-20 : 25	7½	90	29.6	12.2	13,523	0.3125
BT-3 : 26	0	85	20.7	10.0	5,250	0.2500
BT-3 : 27	0	85	20.7	10.0	5,250	0.2500
BT-20 : 28	9½	95	30.6	12.2	14,453	0.3125
BT-Spec.: 29	0	110	37.5	9.0	21,759	0.3125
BT-8DC : 30	7½	100	29.5	9.0	12,014	0.2500
BT-8DC : 31	0	95	28.4	9.0	11,096	0.2500
BT-45 DE: 32	45	100	46.7	9.0	47,313	0.4375
BT-90HDE: 33	DE	53	xxx	xxx	xxx	xxx

NOTE: Structure No. 33 is located at Billerica #70 Substation

for the conductor and shield wire respectively. The length of the suspension insulator string is 4.5 feet.

The longitudinal stiffness of each pole is the primary parameter to be included in the dynamic model, and in particular, the stiffness-influence coefficients at both shield wire and conductor attachment points are to be reproduced in the model scale.

In the course of this investigation, a rapid method for calculating the stiffness-influence coefficients has been developed. As this method is believed to be suitable for much broader applications beyond the present study, it is described in detail so that anyone interested in computing influence coefficients (either stiffness or flexibility) for tapered steel poles can obtain results rapidly.

2.1 Rapid Method for Obtaining Influence Coefficients of Tapered Steel Pole Transmission Structures

In the design of steel pole type transmission structures the loading conditions are normally specified by the utility customer. The pole manufacturer then develops a design based on the specified loading. Once critical loading conditions are determined, the pole is sized according to the critical loads.

For single-pole type of structures, the resulting design is shown by calculations to have sufficient strength at each section by comparing the design bending moments to the allowed bending moments based on the yield strength (psi) of the material. Sometimes this is done by comparing section furnished versus section required (bending modulus) data. In

addition, the manufacturer also supplies information on the deflection of the pole at various heights under the maximum design loadings.

The present analysis will show that the above design calculations already furnished by the pole manufacturer, may be used to obtain rapid estimates of relative stiffness and flexibility of tapered steel poles.

Consider a tapered steel pole having a cross-section whose shape is doubly symmetric about two orthogonal axes. The cross-section may be either round, hexagonal, octagonal, dodecahedron, etc. The diameter is linearly tapered between D_0 at the base and D_T at the top. The plate thickness may vary by discrete steps from t_0, t_1, t_2 in sequence with the greatest thickness at the base.

Under the specified loading conditions, the applied bending moment, and the allowed bending moment at each section obeys the inequality:

$$M_{app.} < M_{allow.} \quad (1)$$

but the allowed bending moment is also related to section modulus Z and maximum yield stress, f_y ,

$$M_{allow.} = Z f_y$$

where, $Z = CR^2 t$ (2)

in which the local pole radius R , and thickness t , may vary with section height h , above the base plate. The constant C , will depend upon the section shape, but generally is in the range of 2.5 to 3.14.

The radius is graded linearly with section height according to:

$$R = R_0 - \frac{(R_0 - R_T) h}{h_T} \quad (3)$$

where subscripts 0 and T refer to base, top conditions respectively.

The moment applied will depend upon the specified loading tree, but in most cases, the critical bending moment distribution will follow the approximation (See Figure 2A).

$$M_{app.} = M_0 - M_0 \frac{h}{h'} ; \quad 0 < h < h' \quad (4)$$

The moment applied distribution is seen to follow a linear grading with pole elevation up to a height h' , where it then follows a non-linear grading. The pole deflection under the applied moment is also shown in the figure. It follows a non-linear grading up to a pole elevation h' , from which point on it is linear.

The pole deflection y , and the bending moments are related by simple beam theory as follows:

$$\frac{M}{EI} = \frac{d^2 y}{dh^2} \quad (5)$$

where EI is the local bending stiffness of the tapered pole.

2.2 An Approximate Method for Calculating Influence Coefficients of Tapered Steel Poles

Given:

The design bending moments and deflections of tapered steel transmission poles.

Analysis:

In most practical transmission pole structures, the bending moment distribution for design of the pole is approximately a linearly graded function, having its maximum value at the ground line or base. There is a small departure of the linear grading towards the top of the pole

(See Figure 2A). The corresponding deflection shape is a curve approximating a parabola (Figure 2B).

Except for the small region near the top, the bending moment distribution may be represented by a concentrated load at the zero intercept of the linearly graded portion. This value is equal to the ground line moment GLM, divided by the distance to the zero intercept. From Figure 2A, the concentrated load is:

$$P' = \frac{144 \times 100 \times 1/12}{79} = 15.18 \text{ kip} \quad (6)$$

and the corresponding deflection at that point is 69 inches. The total strain energy in the pole due to that load is thus:

$$E = \frac{15.18 \times 1000}{2} \times \frac{69}{12} = 43,670 \text{ ft-lbs} \quad (7)$$

The strain energy associated with the small bending moment distribution near the top can be approximated by a triangular moment distribution having a maximum value of 1,000 inch-kips, spread over a base line from pole height 60 feet to 95 feet. Over that portion of the pole the slope of its deflection curve is nearly constant and equal to (cf. Figure 2B).

$$\text{Top Slope} = \frac{95}{(95 - 40) \times 12} = 0.1439 \text{ radians} \quad (8)$$

The corresponding strain energy due to this distributed moment is:

$$\int M d\alpha = \int M \frac{d\alpha}{dh} = 0 \quad (9)$$

The slope, α , is a constant value over the range $h_T - h''$. Hence, the contribution to total strain energy may be effectively represented by the single equivalent concentrated load P' , at height h' . Under this concept

FIGURE 2A. Bending Moment (100 inches-kips)

FIGURE 2B. Deflection (inches)

the influence coefficients are:

$$\text{STIFFNESS: } k_{00} = \frac{15.18 \times 1000}{69 \times 1/12} = 2,640 \text{ lbs./ft.}$$

$$\text{FLEXIBILITY: } u_{00} = \frac{12}{2,640} = 4.544 \text{ inches/kip} \quad (10)$$

Subscript zero means that loads are applied at zero moment intercept.

Other influence coefficients, for example, k_{01} , u_{01} corresponding to tip deflection due to a load at the concentrated load point, may be found from the deflection curve Figure 2B.

$$\text{STIFFNESS: } k_{01} = \frac{15.18 \times 1000}{95 \times 1/12} = 1,917 \text{ lbs./ft.}$$

$$\text{FLEXIBILITY: } u_{01} = \frac{12}{1,917} = 6.258 \text{ inches/kip} \quad (11)$$

The same procedure may be applied using the deflection curve to find other influence coefficients due to loading at the same point, designated reference point zero (0). Further, by Maxwells' reciprocity theorem, loads, applied at these points yield deflection of point zero directly. For example, the deflection at point zero due to a unit load at the top is 6.258 inches/kip. For analysis purposes, this, however, is not sufficient. What is needed in analyses are the influence coefficients k_{ii} and k_{ij} where both i and j are arbitrary.

2.3 Generalized Influence Coefficients

Consider that the influence coefficients may be obtained for the designated reference point zero as in the preceeding analysis. It is desired to convert information to the general case of a general point i

located anywhere along the pole and separated a distance " Δh " (positive above) from the reference point.

Then the stiffness-influence coefficient k_{ii} is by definition the force required at point i to produce a unit deflection at point i . The load system is considered to consist of two parts: 1) a concentrated load at point zero equal in magnitude to k_{ii} , and 2) a pure couple equal in magnitude to $k_{ii} \Delta h$.

The desired generalization is based on a constant energy hypothesis. That is, the total strain energy in the tapered pole remains constant, even though the force systems differ. Further, for convenience, the total strain energy associated with the deflection under the design loading, and which is normally provided by the manufacturer together with the design moment distribution, is used as a reference.

For example, it has been found that a "design load" of 15.18 kip at a pole height of 79 feet will produce the same strain energy and ground line moment as the loading system used to design the pole. Under the constant energy hypothesis, a load P_1 , applied to the top of the pole will produce strain energy equal to:

$$E = 1/2 \int_0^{h'} M d\alpha + 1/2 \int_0^{h'} P \Delta h d\alpha + \int_{h'}^{h_T} M'' d\alpha \quad (12)$$

$$\text{where, } M = P_1 h' (1 - h/h')$$

$$M'' = P_1 \Delta h \left(1 - \left(\frac{h - h'}{\Delta h} \right) \right)$$

$$\Delta h = h_T - h'$$

$$\Delta h = 95 - 79 = 16 \text{ feet}$$

The first integral has already been calculated for the design loading

$P_D = 15.18$ kip, so that its value in the above is:

$$\frac{1}{2} h' \int_0^{\delta_D} M d\alpha = \frac{P_1 \delta_D}{2} \quad (\delta_D = 69 \text{ inches}) \quad (13)$$

Note that $P_1 \neq P_D$ except under the special case when the load point is at 79 feet pole height.

The second integral is just equal to the product

$$\frac{1}{2} P_1 \Delta h \alpha_D \quad (14)$$

where from Figure 2B, $\alpha_D = 0.1439$ radians. The third integral is zero because the change in slope over the range h' to h_T is zero.

Under the constant energy hypothesis,

$$E = 43,670 \text{ ft-lbs} = \frac{P_1}{2} \left(\frac{69}{12} \right) + \frac{P_1 (16)}{2} (0.1439)$$

$$\text{therefore, } P_1 = 11.05 \text{ kip} \quad (15)$$

To check this, compute:

$$\frac{P_1 \times \text{top deflection}}{2} = \frac{11.05 \times 95}{2 \times 12}$$

which is 43,739 ft.-lbs. compared to 43,670 ft.-lbs. A further check was made by comparing the deflection at pole top due to a kip load.

approximate analysis: $95/11.05 = 8.597$ inches/kip

"exact" analysis: $= 8.600$ inches/kip

The "exact" analysis used the classical beam theory and the bending stiffness distribution corresponding to the moment of inertia for the do-decahedron cross section. The agreement between the two methods is excellent.

3. STIFFNESS INFLUENCE COEFFICIENTS - FULL SCALE

3.1 Calculation of Influence Coefficients by Constant Energy Method

The foregoing illustration is generalized to obtain the desired influence coefficients:

k_{11} - Pole Top Stiffness

k_{22} - Upper Conductor Point Stiffness

k_{12} - Cross-Stiffness

In all of the above cases the deflection corresponding to the force is unity (by definition of influence coefficient).

The first part of the analysis is tabulated in Table 2.

The second part of the analysis is tabulated in Table 3. The calculation of the load P_i is based on the constant energy hypothesis and is found from:

$$2E_D = \text{constant} = P_D \delta_D = P_i \delta_D + P_i \Delta h_i \alpha_D \quad (16)$$

Where the first term is the load P_i applied at the height x_D , and the second term is the pure couple $P_i \Delta h_i$ applied through the rotation angle α_D .

The third part of the analysis is tabulated in Table 4.

The influence coefficients are, by definition,

$$k_{11} = P_1 / \delta_1$$

$$k_{22} = P_2 / \delta_2$$

$$k_{12} = P_2 / \delta_{12}$$

The foregoing method of calculation has the principal advantage of utilizing

TABLE 2

CALCULATION OF CONSTANT ENERGY*

Pole Number and Type	(1) GLM (M_0) in-kips	(2) $(\frac{\Delta M}{\Delta X})_0$ (in-kip/ft)	(3) x_D (ft)	(4) P_D (kip)	(5) δ_D (in)	(6) $2E_D$ (ft-lb) x 10^{-3}
23(BT-3)	5,250	72.24	72.67	6.02	74.52	37.38
24(BT-3)	5,250	72.24	72.76	6.02	74.52	37.38
25(BT-20)	14,453	183.9	78.59	15.32	66.89	85.39
26(BT-3)	5,250	72.24	72.67	6.02	74.52	37.38
27(BT-3)	5,250	72.24	72.67	6.02	74.52	37.38
28(BT-20)	14,453	183.9	78.59	15.32	66.89	85.39
29(BT-Sp)	21,759	343.8	63.28	28.65	33.63	80.29
30(BT-8)	12,014	183.6	65.43	15.30	43.00	54.82
31(BT-8)	12,014	183.6	65.43	15.30	43.00	54.82
32(BT-45)	47,313	711.3	66.50	59.28	33.34	167.7

*NOTES:

- 1) GLM = M_0 obtain from computer print-out supplied by pole manufacturer (See sample, Figure 2A)
- 2) Computed from groundline moment gradient supplied by pole manufacturer
- 3) Computed from $x_D = M_0 / (\Delta M / \Delta x)_0$
- 4) Computed from $P_D = M_0 / x_D \cdot 12$
- 5) Obtain from computer deflection print-out supplied by pole manufacturer (See sample, Figure 2B)
- 6) Computed from $2E_D = P_D \times \delta_D \cdot \left(\frac{1}{12}\right)$

TABLE 3
LOADS REQUIRED FOR CONSTANT ENERGY*

Pole No.	(1) x_D (ft)	(2) H (ft)	(3) Δh_1 (ft)	(4) Δh_2 (ft)	(5) α_D (Rad)	(6) P_1 (kip)	(7) P_2 (kip)
23	72.67	85	12.33	2.83	0.1553	4.601	5.622
24	72.67	85	12.33	2.83	0.1553	4.601	5.622
25	78.59	90	11.41	1.91	0.1440	11.82	14.59
26	72.67	85	12.33	2.83	0.1553	4.601	5.622
27	72.67	85	12.33	2.83	0.1553	4.601	5.622
28	78.59	95	16.41	6.91	0.1440	10.76	12.99
29	63.28	110	46.72	37.22	0.1260	9.26	10.71
30	65.43	100	36.57	25.07	0.1284	6.62	8.059
31	65.43	95	29.57	20.07	0.1284	7.43	8.899
32	66.50	100	33.50	24.0	0.1280	23.30	28.15

*NOTES:

- 1) From previous calculation
- 2) Pole Height
- 3) $\Delta h_1 = H - x_D$
- 4) $\Delta h_2 = H - 9.5 - x_D$
- 5) α_D is slope of deflection curve at pole top obtained from manufacturer print-out
- 6), 7) $P_1 = P_D / (1 + \Delta h_1 \alpha_D / \delta_D)$

TABLE 4

BASIC POLE INFLUENCE COEFFICIENT

Pole No.	(1)	(2)	(3)	(4)	(5)	(6)
	δ_1	δ_2	δ_{12}	k_{11}	k_{22}	k_{12}
	(in)	(in)	(in)	(lb/in)	(lb/in)	(lb/in)
23	97.88	79.79	97.88	47.00	70.45	57.43
24	97.88	79.79	97.88	47.00	70.45	57.43
25	86.59	70.08	86.59	136.5	208.1	168.4
26	97.88	79.79	97.88	47.00	70.45	57.43
27	97.88	79.79	97.88	47.00	70.45	57.43
28	95.46	78.69	95.46	112.5	165.0	136.1
29	95.65	80.59	95.65	96.6	132.8	112.0
30	94.43	79.00	94.43	70.1	102.0	85.36
31	86.60	72.00	86.60	85.7	123.6	102.7
32	80.15	64.66	80.15	290.7	435.3	351.3

*NOTES:

- 1), 3) Pole top deflection from computer print-out supplied by manufacturer
- 2) Deflection at upper arm attachment point from computer print-out supplied by manufacturer

data already supplied by the pole manufacturer, and by the earlier comparison to results obtained by independent calculations yields a high confidence level in its application.

A word of caution is in order. That is, the method should not be used to find influence coefficients at points below the pole height x_D because, in general, the slope of the deflection curve at such points differs from the assumed constant value α_D .

3.2 Effects of Other Stiffness Elements

The previous analysis has resulted in the stiffness for each of the steel poles in the system. However, the calculated values apply only to the basic stiffness of the pole shaft, free-standing as a cantilever beam. Additional stiffness elements to be considered include:

- 1) Effects of pole shaft torsion due to forces applied at end of upper conductor arm.
- 2) Effect of pole arm support of upper conductor.
- 3) Effect of lower phase conductors.
- 4) Effects of underbuilt circuit on structures 29, 30, 31, 32.

The first two effects tend to diminish the values already calculated, while the next two effects tend to increase them.

3.3 Pole-Shaft Torsion Flexibility

If a torque T is applied to a tapered pole at some elevation h_2 , the same torque is resisted by the pole cross-section at all elevations below that point. The resistance torque on a symmetrical thin-walled

section is,

$$T = 2\pi \int_0 \sigma_s t R^2 d\theta \quad (18)$$

where σ_s is shear stress on thin wall

t is wall thickness

R is radius of section

In the present case, the section is twelve sided, and is approximated as a circle, leading to:

$$T = 2\sigma_s A t$$

where $A = \pi R^2$ = cross-section area enclosed by perimeter.

Within the elastic range of deformation, the shear stress is related to shear strain γ , by the shear modulus G ,

$$\sigma_s = G\gamma \quad (19)$$

and for two nearby sections separated by a differential height increment, dh

$$dT = 2Gtd(A\gamma) = 0 \quad (20)$$

But, the shear strain and section twist rotation are related by

$$\gamma dh = R d\theta \quad (21)$$

leading to:

$$d(A\gamma) = d\left(\frac{Rd\theta}{dh} \cdot A\right) = 0 \quad (22)$$

from which the angular rotation at height h_2 can be found

$$\theta(h_2) = \int_0^{h_2} \frac{K dh}{R \cdot A} \quad (23)$$

the constant K , is found from the requirement that the rate of twist at

the base (ground line) is,

$$\left(\frac{d\theta}{dh}\right)_0 = \frac{Y_0}{R_0} = \frac{(\sigma_s)_0}{GR_0} = \frac{T}{2A_0 t_0 GR_0} \quad (24)$$

whence,

$$K = \left(\frac{d\theta}{dh}\right)_0 R_0 A_0 \quad (25)$$

and,

$$\theta(h_2) = \left(\frac{d\theta}{dh}\right)_0 \int_0^{h_2} \left(\frac{R_0 A_0}{RA}\right) dh \quad (26)$$

the ratio under the integral is related by taper:

$$\frac{RA}{R_0 A_0} = (1 + B_2 \eta)^3 \quad (27)$$

$$B_2 = -\left(1 - \frac{R_2}{R_0}\right)$$

where,

$$\eta = h/h_2 \quad (28)$$

so that,

$$\theta(h_2) = \frac{T}{2A_0 t_0 G} \left(\frac{h_2}{R_0}\right) \int_0^1 \frac{dx}{(1 + B_2 x)^3} \quad (29)$$

The rotation at pole height h_2 is produced by the load P_2 , acting through the pole arm length L , ($T = P_2 L$), so that the deflection influence coefficient (flexibility) becomes,

$$\frac{\delta_2}{P_2} = \frac{L^2 h_2}{2A_0 t_0 GR_0} \int_0^1 \frac{dx}{(1 + B_2 x)^3} \quad (30)$$

Numerical values for these flexibility influence coefficients are shown in Table 5 for the different pole structures, assuming a pole arm length of seven feet, measured from the pole axis center line to the end of the arm.

3.4 Pole Arm Flexibility

In addition to pole-shaft torsion, the arm which supports the conductors is subject to elastic deformation.

Under an applied longitudinal load at the tip of the arm, the tip will deflect relative to the base, under a bending moment distribution,

$$M = M_0 \left(1 - \frac{x}{\ell}\right) \quad (31)$$

which is seen to be linearly graded. From simple beam theory, the slope of the deflection curve is:

$$\frac{dy}{dx} = \int_0^x \frac{M}{EI} du \quad (32)$$

Because the moment of inertia follows the local radius to a power of three, while the allowed moment follows the section which in turn, follows the local radius to a power of two, the ratio of allowed moment to moment of inertia is:

$$\frac{M_{\text{allow}}}{I} = \frac{I}{R} \cdot f_y \cdot \frac{1}{I} = \frac{f_y}{R} \quad (33)$$

where f_y is the allowed yield stress and R is the local radius of the cross section.

Manufacturers try to match the applied and allowed bending moments closely, so a reasonable assumption is to assume that $M = M_{\text{allow}}$. Under

TABLE 5

FLEXIBILITY DUE TO POLE SHAFT TORSION

Pole No.	(1) h_2 (ft)	(2) t_0 (in)	(3) $2R_0$ (in)	(4) B_2	(5) $f(B_2)$	(6) δ_2/P_2 (in/kip)
23	75.5	1/4	20.7	-0.459	2.631	1.0176
24	75.5	1/4	20.7	-0.459	2.631	1.0176
25	80.5	5/16	29.5	-0.524	3.262	0.3528
26	75.5	1/4	20.7	-0.459	3.262	1.0176
27	75.5	1/4	20.7	-0.459	3.262	1.0176
28	85.5	5/16	30.6	-0.541	3.462	0.3559
29	100.5	5/16	37.5	-0.685	6.626	0.4351
30	90.5	5/16	29.5	-0.629	4.977	0.6052
31	85.5	5/16	28.4	-0.615	4.665	0.6009
32	90.5	7/16	46.7	-0.711	7.725	0.1689

NOTES:

Shear modulus $G = 10^7$ psi

Base Section $A_0 = \pi R_0^2$

5) Taper function $f(B_2) = \int_0^1 \frac{dx}{(1 + B_2 x)^3}$

$$f(B_2) = \frac{1}{2B_2} \left(1 - \frac{1}{(1 + B_2)^2} \right)$$

that assumption, the deflection curve slope is given by,

$$\left(\frac{E}{f_y}\right) \frac{R_0}{\ell} \left(\frac{dy}{dx}\right) = x/\ell \int_0^u \frac{du}{1 + Bu} \quad (34)$$

The tip deflection can be found by integration:

$$\begin{aligned} \left(\frac{E}{f_y}\right) \left(\frac{R_0}{\ell}\right) \left(\frac{\delta}{\ell}\right) &= \frac{1+B}{B^2} \ln(1+B) - \frac{1}{B} \\ &= G(B) \end{aligned}$$

where,

$$B = - \left(1 - \frac{R_T}{R_0}\right)$$

$$G(B) = \frac{1+B}{B^2} \ln(1+B) - \frac{1}{B} \quad (35)$$

The tip load corresponding to the allowed bending moment M_0 , is found from,

$$P = \frac{M_0}{\ell} = \frac{f_y Z_0}{\ell} \quad (36)$$

where Z_0 is the base section modulus.

Though the method is admittedly approximate, it does provide a reasonable estimate of the pole arm stiffness.

In the present case, there are two arms to consider, shown in Figure 3 and Figure 4.

EXAMPLE 1. Calculations Follow for Pole Arm FT 11293

Taper Ratio: $\frac{R_T}{R_0} = \frac{6}{8.312} = 0.6713$

$$G(B) = \frac{1+B}{B^2} \ln(1+B) - \frac{1}{B} = 0.5739$$

FIGURE 3. CONDUCTOR ARM-FT11293

FIGURE 4. CONDUCTOR ARM-FT11300

(Example 1, cont.)

arm length,	ℓ	= 72 inches
Young's modulus,	E	= 29×10^6 psi
yield stress,	f_y	= 50,000 psi
Base value,	R_0	= $\frac{5.937}{2} = 2.968$ inches
Base section,	Z_0	= 7.251 in^3
Base moment.	M_0	= $7.251 \times 50 = 362.5$ in-kip
Load,	P	= $\frac{362.5}{72} = 5.035$ kip
Deflection,	δ	= 0.1440 feet
Stiffness,	P/δ	= 2.913 kip/in

Similiar calculations for pole arm FT 11300 yield the results:

$$\frac{P}{\delta} = 4.855 \text{ kip/inch}$$

These stiffness values, or their inverse which is flexibility, may be combined with earlier results as shown in Table 6.

3.5 Generalizations

The foregoing analysis method may be extended to include a wider class of tapered beams.

It is well-known that the stiffness of a straight uniform beam for a transverse load applied to distance ℓ , from the base is,

$$k = \frac{3EI}{\ell^3} \quad (37)$$

It remains to normalize the previous results against this well-known reference, but including arbitrary taper and arbitrary cross-section.

TABLE 6

INFLUENCE COEFFICIENTS INCLUDING THE
EFFECTS OF POLE ARM AND POLE TORSION FLEXIBILITY

Pole No.	k_{11}	$\frac{\text{lb}}{\text{in}}$ k_{12}	k_{22}
23	47.0	57.43	64.30
24	47.0	57.43	64.30
25	136.5	168.4	181.78
26	47.0	57.43	64.30
27	47.0	57.43	64.30
28	112.5	136.6	147.9
29	96.6	112.0	120.4
30	70.1	85.36	93.02
31	85.77	102.7	110.7
32	290.7	351.3	374.7

The flexibility of the tapered beam has been found from previous results (cf. Equations 34, 35 and 36).

$$\delta/P = G(B) \frac{\ell^2 f_y}{E R_0} \times \frac{\ell}{f_y Z_0}$$

$$\delta/P = \frac{G(B) \ell^3}{E I_0} \quad (38)$$

where I_0 is the base section moment of inertia. Inverting this to obtain stiffness, and normalizing by the expression for the uniform cantilever beam stiffness yields

$$\frac{P/\delta}{k} = \frac{1}{3G(B)} = \lambda \quad (39)$$

The parameter λ is defined as the stiffness ratio of the tapered beam normalized to the uniform beam. The stiffness ratio is thus seen to be a function only of taper ratio. Some typical values are tabulated as follows in Table 7. Taper ratio is defined as the diameter at length ℓ divided by the base diameter. Estimates of tapered beam stiffness may be obtained rapidly by calculating the stiffness of a uniform beam having the base section moment of inertia of the tapered beam and applying the ratio λ .

3.6 Effects of Lower Conductors

In the dynamic test model, the overhead shield wire and the uppermost conductor are to be simulated for dynamic motions. An assumption is that the lower conductors would not participate dynamically in the resulting loading of the structure. However, because pole flexibility induces deflections at points below the upper phase conductor, there is a static

TABLE 7

STIFFNESS RATIO OF TAPERED CANTILEVER BEAMS

<u>Taper Ratio</u>	<u>Stiffness Ratio, λ</u>
0.3	0.4813
0.4	0.5108
0.5	0.5420
0.6	0.5693
0.7	0.5858

contribution to structure stiffness due to restraint provided by the lower conductor attachments to the pole.

Under an assumption that all lower conductors remain motionless under longitudinal displacement of the pole, there is an additional stiffness element at each conductor below the upper phase, due to the longitudinal swaying of the suspension insulator.

A photograph of structure number 24 viewed with number 25 in the background is shown in Figure 5. Consider a suspension insulator, having a small longitudinal displacement at the top as shown in the accompanying sketch.

The longitudinal force required for static equilibrium is found from,

$$\Delta F \cdot b = (W_{12} + W_{ST}) b \theta + W_I \frac{b}{2} \theta \quad (40)$$

FIGURE 5. Type BT-3 and BT-20 Steel Pole

and since,

$$b\theta = \Delta x$$

the equivalent local stiffness is

$$k_0 = \frac{\Delta F}{\Delta x} = \frac{W_{12} + W_{ST} + W_{I/2}}{b} \quad (41)$$

the effect is similar to that which would occur if a spring of stiffness k_0 had been attached to the pole at the local lower conductor suspension point, and anchored to ground. Thus, any longitudinal displacement of the pole at those points would require energy storage equal to

$$E_0 = \frac{1}{2} k_0 (\Delta x)^2 \quad (42)$$

The middle phase conductor is suspended from a point which is 22 feet below the top of the pole. While the lower phase conductor suspension point is 32.5 feet below the top. The shape of the pole deflection curve may be used to determine the respective values for Δx displacement.

Additional energy will be required if friction is present in the insulator strings and hardware. This has been accounted for by arbitrarily increasing the span weight W_{12} by a factor of 10 percent.

For convenience, the pole deflection shape previously used to compute the influence coefficients of the basic pole shaft, and identified as the Constant Energy Hypothesis was used. The calculations are listed in Table 8. There are no suspension insulators on structure Number 32, being a heavy 45 degree dead-end.

The additional strain energy, due to insulator swing, represented by the last column of figures may be compared to the basic strain energy of the pole shaft under the same deflection shape shown in Table 9.

TABLE 8

CONTRIBUTION TO STRAIN ENERGY DUE TO LOWER CONDUCTORS

Structure Number	Insulator length 4.5 feet, Strut weight 100 lbs.						
	W_{12} (lb)	W_{ST} (lb)	$W_{I/2}$ (lb)	k_0 (#/ft)	δ_3^2 (ft ²)	δ_4^2 (ft ⁴)	$k_0(\delta_3^2 + \delta_4^2)$ (ft-lb x 10 ⁻³)
23	1165	0	41	268	22.6	11.2	9.058
24	1165	0	41	268	22.6	11.2	9.058
25	1029	100	41	260	17.4	8.77	6.804
26	909	0	41	211	22.6	11.2	9.058
27	919	0	41	213	22.6	11.2	9.058
28	909	100	41	233	23.2	12.34	8.278
29	889	0	41	206	27.3	16.92	9.109
30	945	100	41	241	25.0	14.7	9.565
31	1014	100	41	256	18.7	11.11	7.649

NOTE: δ_3 = middle phase; δ_4 = bottom phase

TABLE 9

COMPARISON TO BASIC STRUCTURE STRAIN ENERGY

Structure Number	23	24	25	26	27	28	29	30	31
$\frac{k_0(\delta_3^2 + \delta_4^2)}{10^3}$	9.058	9.058	6.804	9.058	9.058	8.278	9.109	9.565	7.649
$\frac{2ED}{10^3}$	37.38	37.38	85.39	37.38	37.38	85.39	80.29	54.82	54.82
Percent Increase	24.23	24.23	7.96	24.23	24.23	9.69	11.34	17.44	13.95

The percentage increase shown by the last set of figures must be applied as an increment to the basic pole shaft stiffness influence coefficients.

An increment applies to all structures listed in Table 9. In addition, however, structures 29, 30, 31, and 32 support the underbuilt circuit. Accordingly, the stiffness of those structures must be modified to include the underbuilt contributions.

3.7 Underbuilt Stiffness

As in the preceeding analysis, the additional secondary circuit below the primary 115 Kv circuit contributes to the total stiffness of structures 29 through 32. The underbuilt 23 Kv circuit employs the same conductor (Condor) in three phase. The cover sheet photo shows this circuit on structure 32. For simplicity, it is assumed that all three phases of the underbuilt are lumped together at the same distance from the respective pole tops. The sketch below illustrates this:

The distance from top of the pole to the underbuilt attachment points are:

Structure Number	29	30	31	32
Distance From Top	54.5'	48.5'	48.5'	52'

The insulator length at the suspension points is assumed to be one foot (2 discs).

Following the same procedure previously explained for the lower conductors in the primary circuit, the suspension point stiffness for structures Number 30 and 31 respectively, are $k_0 = 957 \text{ lb/ft}$, and $k_0 = 1,026 \text{ lb/ft}$.

3.8 Catenary Stiffness

In any conductor span, especially one that is anchored solidly by dead-ending, the restraint offered by the catenary itself is a non-linear function of displacement of the dead-end in the longitudinal direction. For small displacement, this "catenary stiffness" may be approximated from the known static tension and the slack in the span corresponding to that tension.

A well-known expression for slack is,

$$x_0 = \frac{8}{3} \frac{d_0^2}{\text{span}} \quad (43)$$

where

x_0 = slack

d_0 = catenary sag at midspan

span = span length

If the dead-end is subject to a small incremental displacement in the longitudinal direction, the other end remaining fixed, then the effective stiffness of the catenary is found to be

$$k_c = T_0/2x_0 \quad (44)$$

where

T_0 = initial static tension.

In the three spans between structures Number 29 and 32, the numerical values are as follows (for $T_0 = 5000$ lbs.) in Table 10. The equivalent stiffness system, including the suspension stiffness k_0 and the catenary stiffness k_c is shown below

For one conductor

Stiffness (kip/ft)

For three conductors

TABLE 10
 CANTENARY STIFFNESS
 ($T_0 = 5000$ lbs)

Span	Span Length (ft)	Sag (ft)	x_0 (ft)	k_c kip/in
29 - 30	360	7.790	0.402	.4509
30 - 31	415	10.50	0.708	.2942
31 - 32	417	10.60	0.718	.2901

These stiffness elements are combined to obtain the net stiffness at each pole due to the underbuilt circuit.

For Structure Number 29

$$\text{Flexibility} = \frac{1}{16.23} + \frac{1}{10.59} + \frac{1}{10.44} = 0.251 \text{ kp/ft.}$$

$$\text{Stiffness} = \frac{1}{0.251} = 3.971 \text{ kp/ft}$$

For Structure Number 30

$$\begin{aligned} \text{Catenary Stiffness} &= 16.23 + \left(\frac{1}{10.59} + \frac{1}{10.44} \right)^{-1} \\ &= 16.23 + 5.260 = 21.49 \text{ kp/ft} \end{aligned}$$

$$\text{Net Stiffness} = \left(\frac{1}{21.49} + \frac{1}{2.87} \right)^{-1} = 2.57 \text{ kp/ft}$$

For Structure Number 31

$$\begin{aligned} \text{Catenary Stiffness} &= 10.44 + \left(\frac{1}{10.59} + \frac{1}{16.23} \right)^{-1} \\ &= 10.44 + 6.41 = 16.85 \text{ kp/ft} \end{aligned}$$

$$\text{Net Stiffness} = \left(\frac{1}{16.85} + \frac{1}{3.07} \right)^{-1} = 2.58 \text{ kp/ft}$$

Structure Number 32 is considered as a special case because of the large angle and because of its proximity to the solid dead-end.

The above net stiffness values are used to obtain the final contribution by the underbuilt. To illustrate, Structure Number 29 is worked out in detail.

EXAMPLE 2. Structure Number 29

Assume all three underbuilt conductors located at 54.5 feet from

pole top or at a height of (110-54.5)feet. The deflection at that height under the basic design loading (Constant Energy Hypothesis) is found from the pole manufacturer computer print out to be 25.55 inches. Apply the net underbuilt stiffness of 3.971 kp/ft to that deflection and obtain:

$$3.971 \left(\frac{25.55}{12} \right)^2 = 18,000 \text{ft-lbs}$$

as the energy contribution. For this pole the basic energy $2E_D$ was found to be $2E_D = 80,290 \text{ ft-lbs}$. Therefore, the underbuilt adds $18,000/80,290$ or 22.42 percent to the strain energy.

Previous calculations for the lower conductor of the primary circuit yielded an addition of 11.34 percent to the basic pole strain energy. Hence, the total increment of strain energy contributed by all lower conductors is 33.76 percent.

The underbuilt calculations are summarized in Table 11. Combining the results from Table 11 with earlier calculations leads to the final influence coefficients values shown in Table 12.

EXAMPLE 3. Structure Number 32

This structure is a heavy 45 degree angle having all conductors attached in strain, and facing a relatively short span to structure Number 33, a solid dead-end.

The stiffness elements are broken down as follows:

1) Basic Pole:

$$k_{11} = 3.488 \text{ kp/ft}$$

$$k_{22} = 5.223 \text{ kp/ft}$$

$$2E_D = 164,700 \text{ ft-lb}$$

TABLE 11

STRAIN ENERGY CONTRIBUTION

Structure Number	Height (ft)	Distance From Top (ft)	Net Stiffness (kp/ft)	Basic Deflection (in)	Percent Strain Energy
29	110	54.5	3.971	26	22.42
30	100	48.5	2.570	27	23.71
31	95	48.5	2.580	23	17.28

TABLE 12

INFLUENCE COEFFICIENTS INCLUDING THE EFFECTS OF POLE ARM AND POLE
TORSION FLEXIBILITY AND THE EFFECTS OF LOWER CONDUCTOR STIFFNESS(lb./in.)

Pole Number	k_{11}	k_{12}	k_{22}
23	58.37	71.32	78.24
24	58.37	71.32	78.24
25	147.28	181.7	194.1
26	58.37	71.32	78.24
27	58.37	71.32	78.24
28	123.4	149.3	160.6
29	129.9	150.6	156.8
30	98.91	120.4	126.6
31	112.5	134.7	140.6

2) Catenary Stiffness (262 ft. span)

$$k_c = 14.0 \text{ kp/ft/conductor}$$

3) Pole Arm Stiffness = 58.3 kp/ft

4) Strain Energy Breakdown (ft-lbs)

i) Basic Pole (2ED)	164,700
ii) Lower Conductors (3 + 4)	327,000
iii) 3 Underbuilt Conductors	82,000

The stiffness properties of this pole in the system are thus seen to be dominated by the very large catenary stiffness of the short span between structures 32 and 33.

For modeling purposes, since the catenary stiffness element is non-linear, it was considered more exact to reproduce the actual catenary spans for all conductors.

4. MODEL DESIGN AND INSTRUMENTATION

In designing the model the following objectives were set:

- The model should represent dynamic behavior of the line from (i) broken conductor, (ii) broken static wire, (iii) broken insulator, (iv) ice unloading, and (v) galloping.
- The conductor and static wire mass per unit length shall represent the heavy load condition (0 degree F, $\frac{1}{2}$ in ice).
- The conductor and static wire tension shall represent the heavy loading condition (3,000 lb static, 5,000 lb. conductor)
- The model stiffnesses shall represent the actual (calculated) stiffness including (i) pole bending, (ii) pole torsion, and arm bending.
- Represent the dynamic coupling action between the static wire and the upper-most conductor phase wire, including the static coupling due to the lower phase wires and the underbuilt phase wires.
- Measure the longitudinal loading at selected structures.

The major part of the model design is concerned with the matching of the flexibility coefficient matrix between the full-scale and the model structures.

Considering that point number (1) is at the pole top static wire attachment, and that point number (2) is at the pole arm tip of the

upper phase, the influence coefficient matrix,

$$u = \begin{bmatrix} u_{11} & u_{12} \\ u_{21} & u_{22} \end{bmatrix} \quad (45)$$

must be matched between model and full scale. The inverse of the u matrix is the stiffness matrix. In both matrices, the main diagonal elements are different, but the off-diagonal elements are equal (Maxwell's Reciprocity Theorem). In the case of flexibility matrix u , the terms are increased by the scale ratio, N , while in the case of the stiffness matrix, k , the terms are reduced by the scale ratio, N . Hence the matrix product, uk , is equal to the unit matrix regardless of the scale.

The principles of similitude thus demand that the relative values of the coefficients in the model match term-for-term the relative values of the full-scale coefficients, and the numerical values between model and full-scale are in the ratio of the scale factor, N . In the present case, it is required to adjust three independent parameters in the model.

Suppose that another phase conductor was to be included in the model similitude scaling. Then the above matrix would be a 3×3 instead of a 2×2 . In that case, it would be required to match six coefficients by adjustment of six independent model parameters. If all three phases were included, the number of independent model parameters increases to ten. The growth in complexity of the model design is readily apparent.

If the natural frequency of the structure was found to be equal to or less than the catenary natural frequency, then in addition to the stiffness/flexibility matrix matching, there would be an additional

requirement to match the mass matrix representing the pole mass distribution. The model design in that case becomes a major undertaking, requiring extensive computation. Fortunately, the full-scale line selected for modelling may be represented by a relatively simple model.

Neglecting the natural frequency of the structure was not done arbitrarily. An analysis was carried out for a typical structure (pole Number 28, type BT-20). In the analysis, the mass distribution of the tapered pole was obtained from manufacturer's data, and the stiffness properties of the pole considered as a free-standing cantilever beam were combined in a classical analysis known as the Rayleigh Method. The lowest natural frequency for the pole was found to be 1.16Hz. This was then compared to the lowest natural frequency of a conductor span (Ruling Span = 400 ft.) which was found to be 0.32 Hz, for a frequency ratio of 3.6 to one.

Because the objective of the present study is to examine the effect of flexibility on dynamic loading, simulation of pole natural frequency was not required. Therefore, the model "pole" natural frequencies were arbitrarily shown to be at least five times the lowest model conductor span frequency. This is believed to give added assurance that the dynamic loads in the model are not influenced by pole mass or frequency ratio.

EXAMPLE 3. Illustrative Example

Structural model similitude can best be illustrated by an example problem.

Both structure Number 25 and structure Number 28 employ the pole design designation, BT-20. The first is a pole 90 feet high and the second is a pole 95 feet high. The influence coefficients for the latter are:

$$\begin{array}{ll} k_{11} = 123.4 \text{ lb/in} & u_{11} = 8.10 \text{ in/kip} \\ k_{22} = 160.6 \text{ lb/in} & u_{22} = 6.23 \text{ in/kip} \\ k_{12} = 149.3 \text{ lb/in} & u_{12} = 6.70 \text{ in/kip} \end{array}$$

Assume that these coefficients are to be matched in the model for a scale factor of $N = 50$. The model coefficients become:

$$\begin{array}{ll} k_{11} = 2.468 \text{ lb/in} & u_{11} = 0.405 \text{ in/lb} \\ k_{22} = 3.212 \text{ lb/in} & u_{22} = 0.311 \text{ in/lb} \\ k_{12} = 2.986 \text{ lb/in} & u_{12} = 0.334 \text{ in/lb} \end{array}$$

The model structure can be designed as a cantilever beam having a constant bending stiffness, EI , a Height, h , and a support point for the conductor a distance, b , above the base. These are three independent parameters available to match the influence coefficients. The relations for such a model are available from textbooks on structures.

$$k_{11} = 3EI/h^3 \quad (46)$$

$$k_{22} = 3EI/b^3 \quad (47)$$

$$k_{12} = 6EI/b^3 \left(\frac{3h}{b} - 1 \right) \quad (48)$$

For a beam of constant EI the ratio of k_{11}/k_{22} is related to the length ratio b/h , or,

$$k_{11}/k_{22} = (b/h)^3 = 2.468/3.212 \quad (49)$$

whence,

$$b/h = 0.916$$

A matrix relationship may now be established:

$$3EI/h^3 \begin{bmatrix} 1 & 1.140 \\ 1.140 & 1.301 \end{bmatrix} = 2.468 \begin{bmatrix} 1 & 1.209 \\ 1.209 & 1.301 \end{bmatrix} \quad (50)$$

Comparing term-for-term, we find that the main diagonal terms are matched, but the off diagonal terms differ by about 5 percent.

Assuming that the above is a sufficiently close match, the size of the model may be determined from the requirement:

$$3EI/h^3 = 2.468$$

or,

$$EI = 0.822h^3$$

If the model is to be fabricated from aluminum bar stock, then

$$E = 10^7 \text{ psi}$$

$$I = dt^3/12 \quad (51)$$

where t is the thickness of bar stock. For $d = 2$ inches

$$(t/h)^3 = 4.93 \times 10^{-7}$$

and

$$t/h = 0.00790$$

Practical considerations dictate the maximum length of the model. Assume that a maximum model height of 16 inches is desired, the $t = 0.126$ inches,

and $d = 2$ inches. Commercially available bar stock is listed with thickness of 0.125, 0.1875 and 0.250.

Final selection among these values is also dictated by the elastic stability of the beam subject to Euler column loading due to the dead weight of the static wire and conductor models. For a fixed cantilever, the Euler buckling load is given by,

$$P_c = \pi^2 EI / 4h^2 \quad (52)$$

and for the above particular design, $EI = 3,366 \text{ lb-in}^2$. So that, the Euler buckling load is,

$$P_c = 32.8 \text{ lb.}$$

A rough estimate of the dead weight of static wire, conductor, and 1/2 inch ice for a span of 400 feet is the following:

Conductor Weight	1.1 lb/ft
Static Wire	0.56 lb/ft
Conductor Ice	1.0 lb/ft
Static Ice	<u>0.51 lb/ft</u>
Total	3.17 lb/ft

Total for a 400 foot span1,268 lb

Now the model scale factor of $N = 50$ applies to both the weight per unit length and the span length. Hence, the model span dead weight will be $1,268/2,500 = 0.507 \text{ lb}$. This is less than the computed critical buckling load.

The selection of the thickness t may now be made based on the nearest

commercial stock available, or $t = 0.125$. This results in the design:

$$h = 16.00 \text{ in.}$$

$$b = 14.6 \text{ in.}$$

$$P_c = 32.8 \text{ lb.}$$

$$EI = 3,366 \text{ lb-in}^2$$

Following this procedure the final model designs for structures number 23 through number 31 are shown in the accompanying Figure 6. The table identifies the specific model number in relation to the structure number. For convenience, the conductor height, b , is constant at 15 inches in all models.

4.1 Structure Number 32

It has already been mentioned that the large angle structure, being next to the hard dead-end, and having a short span between, is dominated in stiffness behavior by the catenary stiffness of the conductors and shield wire.

In the model, a design was selected which would reproduce at model scale the influence coefficients k_{11} and k_{22} of the basic pole. These values, obtained by the Constant Energy Method, are respectively 290 lb/in and 374 lb/in at full scale.

A round aluminum tubing, having a diameter of $5/8$ inches and a wall thickness of 0.030 inches and a cantilever length of 23 inches was found to simulate the stiffness at the shield wire. The upper conductor was located at 21 inches. A provision was also introduced to add the remaining conductors, if desired, to simulate the hard non-linear stiffness at the structure.

<u>Model Number</u>	<u>Pole Number</u>	<u>Height (in)</u>	<u>d/2 (in)</u>
1	23,24,26,27	16.53	0.540
2	30	16.26	0.875
3	31	16.14	0.971
4	29	16.37	1.100
5	28	15.97	1.068
6	25	16.44	1.340

FIGURE 6. TANGENT MODELS (6061-T6 Aluminum x 1/8 in. Thickness)

4.2 Instrumentation

The model transmission line was set up on the roof of Building Number 37 at Massachusetts Institute of Technology, Cambridge, Massachusetts. A complete set of wooden platforms was installed to support the various structure models, at the scaled (50:1) span intervals. A wooden boardwalk was also installed together with a rope safety barrier. The platform, boardwalk, barrier, and orientation of model structure number 25 is shown in Figure 7A. The assembled model line is shown in Figure 7B. Note that the shield wire is attached directly to the pole top, while the conductor is suspended on "insulators". The conductor, and shield wire, consisted of bead chain having the correct mass per foot for the 1/50 scale model, of 20 gms/ft and 10 gms/ft respectively. The scaled model tension was 2.0 pounds and 1.2 pounds in conductor and shield wire respectively. The scale factor for tension is $(50)^2 = 2,500$. The "ground line" of each structure is located three inches above the platform. That ground line was established as a rigid reference, by means of two opposing aluminum angle members shown in Figure 7B bolted to the model structure. The pole top stiffness k_{11} , was checked at each structure before the conductor and shield wires were attached. This was done to verify that the design stiffnesses were actually achieved.

Measurements of loadings at structure number 23 through 27 were accomplished by a method of substituting the angle base support, by a knife edge support shown in Figure 8. That is each of the structures at which measurements were made, were supported in turn by the knife edge

FIGURE 7A. MODEL STRUCTURE NUMBER 25 BEFORE SETUP

FIGURE 7B. MODEL STRUCTURE NUMBER 29 AFTER SETUP
WITH STRUCTURES NUMBERS 30, 31, 32 and
33 IN BACKGROUND

*NOTE: Load-cell beam consists of strain-gage load cells orthogonally oriented on heavy steel beam (55 lbs), see attached photos.

FIGURE 8. MODEL DESIGN

arrangement--the aluminum angles being removed. A rigid steel rod 30 inches by 1/2 inch diameter was attached at the model structure base three inches above the platform. Equal lengths of 15 inches at either side of the structures center line assured that no initial dead weight offsetting moments were applied about the knife edge. A pin joint provided a connection to the sensitive load-cell beam. See Figure 9. Doubler plates of the same thickness as the angles (1/8 inch) were attached to the three inch portion of the structural model. The ground line was the same as before at three inches above the platform. The use of rigid steel rod proved convenient for calibration of the systems by merely hanging known weights at the indicated calibration points.

The main idea is that loads applied to the structure by the system of shield wire and upper phase conductor would be resisted by reaction forces at the pin joint and the knife edge.

4.3 Load-Cell Beam and Recorder

The load cell beam is five feet long, five inches in diameter and weighs approximately 55 pounds. It was supported on the platforms transversely to the line on wooden blocks. The internal structure of the beam, shown in Figure 9 consists of a total of five strain-gage type load cells each rated at 48 ounces maximum. There are three "lift" gages each sensitive to forces parallel to their armatures. There are two "drag" gages mounted at 90 degrees. The drag gages are mounted to a heavy 1 inch by 2½ inch steel beam (5 feet long) and the lift gages are mounted to a free-floating assembly attached through weak flexures to the heavy steel beam

FIGURE 9. Views of Force Transducer Balance System

and also to the drag gage armatures. Thus, a pure "lift force" will produce a signal output from the lift gages and no output from the drag gages; and no output from the lift gages is produced for a pure "drag force". The lift gages are offset to provide a means of taking out a pure couple about the beam axis.

A cut-away photo of the load-cell beam is shown in Figure 10. Here an additional rigid Y-shaped yoke plate has been added. The yoke plate is supported at three points by the three lift gage armatures, and also by two weak flexures to the free-floating assembly. The pin-joint load point is attached to this yoke plate.

All flexures have been designed to have flexural stiffness small in comparison to the strain gage armature stiffness, but permitting axial load transfer along the flexure length. The entire cut-away section shown in the figure "floats" relative to the steel beam.

Because the pin joint is located a finite distance from the plane of the yoke plate, a pure "lift-force" will produce only a signal output in the lift gages, but a pure "drag force" will produce signal outputs in both channels. This is so because of pure "drag" force applied to the pin joint also produced a couple which is resisted by the lift gages.

Because of geometric constraints, the load-cell beam was oriented on the platform such that the "drag" gage armatures pointed up. The recorder was a Model Number 321 Hewlett Packard two channel strip chart recorder, with strain gage conditioning included. Power source is 110v ac.

FIGURE 10. LOAD CELL BEAM

5. CALIBRATION PROCEDURES

Several methods of calibration were used to establish the correct relationship between the observed recorder response and the pin-joint reactions H and V. These are defined under the following type descriptions:

- Type I - Loads applied at the end of the rigid steel rod to the vertical load calibration point, no wires attached.
- Type II - Loads applied to the top of model structure with no shield wire or conductor attached.
- Type III - Loads applied directly to the pin joint in a direction parallel to the lift gage armatures.
- Type IV - Loads applied to the pin joint in a direction parallel to the drag armatures.

In all testing, the output of lift gage number 3 was identified as "H channel" and the output of drag gage number 4 was identified as "V channel". The remaining gages were not recorded. They do of course, carry a share of load applied at the pin joint.

Type I Calibration:

A sample recording of a Type I calibration is shown in Figure 11 for a calibration load of 31.4 ounces. The upper trace is the output of "H channel" and the lower trace is the output of "V channel". The zero load point is indicated. Similar records were obtained for loads of 8.2, 15.0, and 21.1 ounces. The calibration constants, obtained by plotting the

Sample Chart Record
 $Q = 31.4$ (oz)

FIGURE 11. TYPE I CALIBRATION RESULTS

trace deflections δ_H and δ_V are shown in the figure.

Notice that under negative loading (up lift at the calibration point) there is a pronounced non-linearity in the δ_H calibration curve. Knife edge lift-off actually occurred at an uplift of about 20 ounces, which would agree with the dead weight (40 ounces) of the structure, steel rod, plate doubler, nuts, bolts, etc.

By convention, δ_H and δ_V will be taken positive for the directions shown in the figure. The 60 Hz timing marks are seen at the bottom of the sample chart record. Timing marks are at one second intervals.

Type II Calibration:

A sample recording of a type II calibration is shown in Figure 12 along with the plotted data for other loads. Positive trace deflection in each channel is associated with a calibration load Q , applied to the pole top away from the load-cell beam.

For these calibration loads there is a slight non-linearity in the δ_H calibration curve, indicating a possible dead-band at 10 w values of Q .

Type III Calibration;

A sample recording of Type III calibration is shown in Figure 13 along with the data for other loads. Notice that there is no chart deflection δ_V , as expected, because only the "lift" gages are loaded. By convention, a positive load is away from the load-cell beam.

Under this type of calibration, two methods were used. In the first method, known weights were hung directly to the pin joint, the load-cell beam being rotated 90 degrees so the "lift" gage armatures pointed down.

FIGURE 12. TYPE II CALIBRATION RESULTS

FIGURE 13. TYPE III CALIBRATION RESULTS

In the second method, the load-cell beam was in its normal operating position with the lift gage armatures aligned with the horizontal, and known weights were attached over a pulley block to a line fastened to the end of the pin joint. The results were identical.

Type IV Calibration:

A sample recording of Type IV calibration is shown in Figure 14 together with the plotted data for the horizontal and vertical channels. Known weights were hung from the pin joint directly, showing the effect of cross-talk in the horizontal channel due to the moment applied to the "lift" gages. Then another set of measurements was made with the loading applied only to the "drag" gages via a special hook on the floating assembly, thus bypassing the lift gages. In the latter, as expected, there was no output in the horizontal channel, and the output in the vertical channel was indistinguishable from that shown in Figure 14.

In Type IV Calibration the sense of the applied load is positive up.

5.1 Comparison of Calibration Types

The sense of the applied loads H and V at the pin joint is such that positive H corresponds to loads away from the load-cell beam, and positive V is up. Under this convention the various calibration constants are summarized:

<u>Calibration Summary (division/ounces)</u>				
Type Calibration	I	II	III	IV
Horizontal Channel	1.0	1.83	0.90	0.60
Vertical Channel	0.70	0.79	0.0	0.78

FIGURE 14. TYPE IV CALIBRATION RESULTS

5.2 Interpretation of Calibration Constants

The basic calibration constants are those under Type III and Type IV calibrations. These may be interpreted as:

$$\text{Type III} \quad \frac{\partial \delta_H}{\partial Q} = \frac{\partial \delta_H}{\partial H} = 0.90$$

$$\text{Type IV} \quad \frac{\partial \delta_H}{\partial Q} = \frac{\partial \delta_H}{\partial V} = 0.60$$

$$\frac{\partial \delta_V}{\partial Q} = \frac{\partial \delta_V}{\partial V} = 0.78$$

In general, the recorder trace deflections may be expressed in terms of the reaction forces H and V, that is:

$$\begin{aligned} \delta_H &= (0.90) H + (0.60) \cdot V \\ \delta_V &= (0.78) \cdot V \end{aligned} \quad (53)$$

From which the reaction forces may be expressed in terms of the trace deflections, or:

$$\begin{bmatrix} H \\ V \end{bmatrix} = \begin{bmatrix} 1.111 & -0.8547 \\ 0 & 1.282 \end{bmatrix} \begin{bmatrix} \delta_H \\ \delta_V \end{bmatrix} \quad (54)$$

Under Type II calibration, the loading system is shown in the accompanying sketch:

Setting up the equations of statics,

$$Q \times 19.5 - H \times 3 - V \times 15 = 0 \quad (55)$$

$$Q + F - H = 0 \quad (56)$$

$$Q = 0.7692 \cdot V + 0.1538 \cdot H$$

$$f = H - Q$$

$$f = 0.8461 \cdot H - 0.7692 \cdot V$$

Now, according to the Type II calibration, if we put $Q = 1$, then,

$$\delta_H = 1.83$$

$$\delta_V = 0.79$$

and from the matrix equation (equation 54),

$$V = 0.79 \times 1.282 = 1.012$$

$$H = 1.111 \times 1.83 - 0.8547 \times 0.79$$

$$= 2.033 - 0.6752$$

$$= 1.357$$

$$f = 0.357$$

subsequently, from the equation of statics:

$$Q = 0.7692 \times 1.012 + 0.1538 \times 1.357$$

$$= 0.7784 + 0.2087$$

$$= 0.987$$

$$f = 0.8461 \times 1.357 - 0.7692 \times 1.012$$

$$= 1.1481 - 0.7784$$

$$= 0.369$$

These results show that the vertical force channel is a better indication of pole top forces than the horizontal channel due to the presence of the horizontal force reactions at the knife edge.

Under Type I calibration, the loading system is shown in the accompanying sketch.

Setting up the equation of statics,

$$Q \times 15 - V \times 15 - f \times 3 = 0 \quad (57)$$

$$Q = V + \frac{f}{5} \quad (58)$$

Now according to Type I calibration, if we put $Q = 1$, then,

$$\delta_H = 1.0$$

$$\delta_V = 0.70$$

and from the matrix equation (equation 54),

$$V = 0.70 \times 1.282 = 0.897$$

$$f = H = 1.111 \times 1 - 0.8547 \times 0.70$$

$$= 1.111 - 0.598$$

$$f = 0.512$$

And, from the equation of statics,

$$Q = 0.897 + 0.512/5$$

$$= 0.999$$

Again the horizontal channel is seen to be significantly influenced by the horizontal reaction at the knife edge.

The above comparison among the calibration types shows that the calibration matrix provides a faithful indication of the pin joint reactions.

In conclusion, Type I and Type II calibration results agree with the calculated results from the matrix to within 1 percent accuracy, thus lending a high confidence level to subsequent data reduction based on the measured reactions, H and V.

6. TESTING SEQUENCE

Seven different test conditions were employed in the model program

- 1) Broken Conductor in Span 24 - 25
- 2) Broken Shield Wire in Span 24 - 25
- 3) Dropped Insulator at Structure 26
- 4) Ice Unload of Conductor in Span 24 - 25
- 5) Ice Unload of Shield Wire in Span 24 - 25
- 6) Galloping Conductor
- 7) Galloping Shield Wire

For each test condition loads were measured at structures 23, 24, 25, 26 and 27. Two basic system configuration were studied, (i) short insulator and (ii) long insulator.

The short insulator configuration was an exact duplication of the insulator length for the Billerica Full-Scale line as presently constituted or 4.5 feet. The long insulator length was arbitrarily chosen at 7.0 feet full-scale. The later might be considered applicable to a higher voltage in the event that the Billerica line is uprated in voltage at some future date.

1. Broken Conductor Tests

The method employed for these tests was to cause a break at mid-span by cutting the bead chain with a wire cutter. The cut was made at a short splice section having five beads between two standard splice

fittings. In this way, the exact number of beads could be restored to the splice each time, thus assuring the return to the same sag and tension. The sequence was repeated with the load-cell beam located at structure 23 through 27, as already stated. A sample recording of the H and V channel responses is shown in Figure 15, where the measured structure is number 24. Each test was repeated at least once. It can be said, in general, repeatability was excellent throughout the test program.

2. Broken Shield Wire Test

These tests were carried out by cutting the shield wire with wire cutters at mid span, again employing five beads in a fixed splice arrangement, as already described. Again loads were measured at structures 23 through 27. For convenience, the entire set of seven tests were carried out at each structure before moving the load-cell beam to the next. A sample recording of the H and V channel response for this test at structure number 24 is shown in Figure 16.

3. Dropped Insulator

In this test, the insulator attachment to structure number 26 was loosely held in place by hand. Then upon coordination with the recorder operator, the insulator was dropped yielding records of the H and V channel response. A sample record for structure number 24 is shown in Figure 17.

4. Ice Unload of Conductor

In this test, a fine wire whose weight is negligible compared to the

EXPLANATION OF LEGENDS

ON

FIGURES 15 - 21

Top Trace - δ_H (gage 3)

Bottom Trace - δ_V (gage 4)

Timing Marks @ 1 second intervals

Attenuation	50	20	10	5
Record Multiplier	1	0.4	0.2	0.1

Values at t_1 - Peak Value

Values at t_∞ - Residual Value

T_0-1 - Time to first peak (seconds)

T(5 cycles) - Time for 5 cycles of oscillation

	t_1	t_∞	T_{0-1}	Max(p-p)	T(5 cycles)	Attenuation
H	30	16	.18 sec.	35	1.43 sec.	50
V	32	22	.17 sec.	32	1.67 sec.	20

FIGURE 15. RUN #G, BROKEN STATIC 50-20

	t_1	t_∞	T_{0-1}	Period(5 cycles)	Attenuation
H	20	5	0.14 sec.	1.88 sec.	50
V	12	5	0.14 sec.	1.88 sec.	20

FIGURE 16. RUN #3, BROKEN CONDUCTOR 50-20

	t_1	t_∞	T_{0-1}	Max(p-p)	T(5 cycles)	Attenuation
H	11	4	.20 sec.	18		20
V	3	1	.20 sec.	6		10

FIGURE 17. RUN #8, DROPPED INSULATOR 20-10

weight of either the conductor or the shield wire was fastened to the bottom of the insulator at structures number 24 and 25. The wire was then tensioned up to the point that the sag in span 24 - 25 increased from its normal value of 2.44 inches to 5.40 inches (10 feet to 22 feet full scale). Then the fine wire was cut and the loads recorded. A sample record at structure 24 is shown in Figure 18.

5. Ice Unload of Shield Wire

Using the same fine wire as above, attachment at each end of the shield wire was made by small hooks. The fine wire was tensioned, increasing the sag in the shield wire from its nominal value of 2.00 inches to 4.87 inches (8.33 feet to 20.3 feet full-scale). After cutting the fine wire, the dynamic loads were recorded. A sample record at structure number 24 is shown in Figure 19.

6. Galloping Conductor

The tests were carried out by exciting the conductor span 24 - 25 by hand at the first natural frequency, or single loop mode. Amplitude control was very rough, but attempts to limit the amplitude (peak-to-peak) to twice the value of the sag used in the ice drop tests were partially successful. When that amplitude was reached, the excitation stopped, and the motion allowed to die out. A sample recording for structure 24 is shown in Figure 20.

7. Galloping Shield Wire

These tests were done in the same way as for the galloping conductor except that the double amplitude was limited to twice the ice drop sag for the shield wire. A sample recording for structure number 24 is shown in Figure 21.

	t_1	t_∞	T_{0-1}	Max(p-p)	$T(5 \text{ cycles})$	Attenuation
H	48	21	.12 sec.	48	2.05 sec.	20
V	34	25	.175 sec.	34	2.05 sec.	10

FIGURE 18. RUN #12, ICE UNLOAD (STATIC) 20-10

	t_1	t_∞	T_{0-1}	Max(p-p)	T(5 cycles)	Attenuation
H	12	2	.30 sec.	20	2.60 sec.	20
V	4	1	.30 sec.	6	2.60 sec.	10

FIGURE 19. RUN #10, ICE UNLOAD CONDUCTOR 20-10

	Max(p-p)	T(5 cycles)	Attenuation
H	12	2.50 sec.	20
V	11	2.50 sec.	5

FIGURE 20. RUN #14, GALLOPING CONDUCTOR 20-5

	t_1	t_∞	T_{0-1}	Max(p-p)	T(5 cycles)	Attenuation
H	10	-	.20 sec.	26	2.15 sec.	20
V	3	-	.20 sec.	9	2.20 sec.	10

FIGURE 21. RUN #13, GALLOPING STATIC 20-10

7. ANALYSIS OF TESTS AND RESULTS AT MODEL SCALE

Because of the large volume of data obtained during the test program, it is desirable to present the results in a consistent fashion. Primarily, the matters of interest include:

- 1) Peak unbalanced loading on the structure.
- 2) Variation of peak loading with structure position relative to the disturbance.
- 3) Residual static unbalanced loading on the structure after all dynamic action stops.
- 4) Time periods of dynamic response.
- 5) Identification of the most severe loading among the test conditions.
- 6) Effects of structure flexibility on peak dynamic loading.
- 7) Effects of insulator length on peak dynamic loading.
- 8) Transferral of model test results to full-scale.

A concept which proved to be convenient and consistent is the concept of "equivalent pole top load". Under this concept, the equivalent pole top loading, F_e is that load, which if applied to the top of the structure will produce the measured pin joint reactions H and V . By the geometry of the test model, the equivalent pole top load is,

$$F_e = \frac{1}{1.3} \left(\frac{H}{5} + V \right) \quad (59)$$

The equivalent pole top load is taken to be positive when it is directed

away from the dead-end structure number 22.

The tests were all conducted in the vicinity of structures 24, 25 and 26 while measurements of loads were taken at these structures, plus structures 23 and 27. Of these structures, 23, 24, 26 and 27 are identical, while 25 is a relatively stiff light angle structure. A comparison of loadings on structure 25 with loadings on structure 24 for tests involving the span 24 - 25 provides, therefore, an indication of the effect of different stiffnesses. Also, a comparison of loadings on structures 23 and 26 with the loadings on structures 24 and 25 provides an indication of the effect of structure position relative to the disturbance.

Because of the nature of the tests, only three of seven test conditions show any residual loading: namely, broken conductor, broken shield wire and dropped insulator.

7.1 Analysis

The reaction forces at the pin joint are used as the primary means of reducing all test data. These have already been discussed, and in terms of the raw data output, δ_H and δ_V , the matrix equation Number 54, provides the necessary conversion to H and V.

There is a need, however, to relate these quantities to something more meaningful in terms of the forces transmitted to the structure by the shield wire and conductor wire.

TABLE 13

SUMMARY OF TEST DATA - SHORT INSULATOR

Test	Peak Value				Final Value			
	δ_H	δ_V	H(oz)	V(oz)	δ_H	δ_V	H(oz)	V(oz)
<u>Station Number 23</u>								
(1) Broken Conductor	-6.0	-1.4	-6.08	-1.79	-5.0	-0.4	-5.68	-0.51
(2) Broken Static	-14.8	-4.6	-14.08	-5.89	-7.2	-2.4	-6.72	-3.07
(3) Dropped Insul.	3.6	1.2	3.36	1.54	0.6	0.1	0.64	0.13
(4) Ice Conductor	-3.2	-0.8	-3.20	-1.02	-	-	-	-
(5) Ice Static	-14.8	-3.2	-15.20	-4.10	-	-	-	-
(6) Gallop Cond.(max p-p)	9.2	4.6	7.36	5.89	-	-	-	-
(7) Gallop Static(max p-p)	24.0	5.0	24.80	6.40	-	-	-	-
<u>Station Number 24</u>								
Broken Conductor	-20.0	-4.7	-20.24	-6.02	-7.0	-1.5	-7.20	-1.92
Broken Static	-28.5	-11.4	-25.08	-14.59	-15.5	-7.8	-12.36	-9.98
Dropped Insul.	7.6	1.4	8.0	1.79	1.4	0.2	1.52	0.26
Ice Conductor	-4.0	-0.7	-4.24	-0.90	-	-	-	-
Ice Static	-20.0	-6.4	-18.88	-8.19	-	-	-	-
Gallop Cond.(max p-p)	4.8	1.1	4.88	1.41	-	-	-	-
Gallop Static(max p-p)	10.4	1.8	11.04	2.30	-	-	-	-
<u>Station Number 25</u>								
Broken Conductor	30.0	4.8	32.16	6.14	6.0	0.8	6.56	-1.02
Broken Static	37.0	8.0	38.00	10.24	21.0	5.8	20.56	7.42

7.2 Derivation of Reference Load

A convenient reference load for the model can be identified. This may be defined by reference to the Type II calibration. From previous work in Section 5, it was found that a unit force at the top of the model produced a horizontal reaction of 1.357 and vertical reaction of 1.012. In other words, if the model were rigid, then a broken shield wire having an initial tension of 1.2 pounds (19.2 ounces) would produce a horizontal reaction of 26.05 ounces and a vertical reaction of 19.43 ounces. Summing moments about the knife edge, it is found that the quantity $\frac{H}{5} + V$ contributes to moments about that point, the balancing moment being provided by the shield wire. The question then is, how many units of $\frac{H}{5} + V$ are required to exactly balance moments about the knife edge for a broken shield wire on a rigid structure? The answer to that is, by applying simple statics to the model system, 25 ounces. Thus 25 ounces in terms of $\frac{H}{5} + V$ becomes a reference quantity in the model, and if all test data are reduced to $\frac{H}{5} + V$ in ounces, a direct comparison may be made to the 25 ounces reference. This type of comparison will be found convenient when translating to full-scale quantities later.

7.3 Test Data Results

All data obtained during the test program is shown in Table 13 and Table 14 for both the short insulator and long insulator. The matrix equation (equation 54) was used to obtain H and V from δ_H and δ_V . The maximum dynamic loading, in terms of the quantity $\frac{H}{5} + V$ are identified

TABLE 13 (concl.)

Test	Peak Value				Final Value			
	δ_H	δ_V	H(oz)	V(oz)	δ_H	δ_V	H(oz)	V(oz)
Dropped Insul.	25.0	3.0	27.60	3.84	2.0	4.0	2.08	-0.51
Ice Conductor	7.2	1.0	7.84	1.28	-	-	-	-
Ice Static	28.0	4.0	30.40	5.12	-	-	-	-
Gallop Cond.(max p-p)	12.8	1.7	14.00	2.18	-	-	-	-
Gallop Static(max p-p)	13.6	1.7	14.96	2.18	-	-	-	-
<u>Station Number 26</u>								
Broken Conductor	8.8	2.6	8.48	3.33	2.2	0.9	1.92	1.15
Broken Static	14.4	4.4	13.76	5.63	4.4	1.8	3.84	2.30
Dropped Insul.	6.0	1.3	6.16	1.66	0.8	0.3	0.74	0.38
Ice Conductor	4.0	1.1	3.92	1.41	-	-	-	-
Ice Static	14.4	5.3	13.04	6.78	-	-	-	-
Gallop Cond.(max p-p)	11.0	3.2	10.64	4.10	-	-	-	-
Gallop Static(max p-p)	20.0	5.4	19.68	6.91	-	-	-	-
<u>Station Number 27</u>								
Broken Conductor	8.8	2.0	8.96	2.56	8.0	0.4	9.28	0.51
Broken Static	6.0	1.8	5.76	2.30	4.8	0.8	5.12	1.02
Dropped Insul.	-12.8	-2.2	-13.60	-2.82	-0.4	-0.2	-0.32	-0.26
Ice Conductor	5.2	1.0	5.44	1.28	-	-	-	-
Ice Static	16.0	3.8	16.16	4.86	-	-	-	-
Gallop Cond.(max p-p)	11.6	1.8	12.48	2.30	-	-	-	-
Gallop Static(max p-p)	10.0	1.6	10.72	2.05	-	-	-	-

TABLE 14
SUMMARY OF TEST DATA - LONG INSULATOR

Test	Peak Value				Final Value			
	δ_H	δ_V	H(oz)	V(oz)	δ_H	δ_V	H(oz)	V(oz)
<u>Station Number 23</u>								
Broken Conductor	-9.6	-1.8	-10.08	-2.30	-1.8	-0.4	-1.84	-0.51
Broken Static	-13.4	-3.1	-13.60	-3.97	-5.0	-1.7	-4.64	-2.18
Dropped Insul.	3.4	1.0	3.28	1.28	0.2	0	0.24	0
Ice Conductor	-2.7	-0.25	-3.04	-0.32	-	-	-	-
Ice Static	-9.2	-3.4	-8.32	-4.35	-	-	-	-
Gallop Cond.(max p-p)	11.0	4.0	10.00	5.12	-	-	-	-
Gallop Static(max p-p)	18.0	4.0	18.40	5.12	-	-	-	-
<u>Station Number 24</u>								
Broken Conductor	-21.0	-5.0	-21.20	-6.40	-5.0	-1.0	-5.20	-1.28
Broken Static	-24.5	-13.5	-18.60	-17.28	-12.5	-8.5	-8.20	-10.88
Dropped Insul.	7.2	2.8	6.40	3.58	0.6	0.1	0.64	0.13
Ice Conductor	-3.2	-0.4	-3.52	-0.51	-	-	-	-
Ice Static	-15.5	-4.5	-15.00	-5.76	-	-	-	-
Gallop Cond.(max p-p)	5.2	1.2	5.28	1.54	-	-	-	-
Gallop Static(max p-p)	12.5	0.6	14.52	0.77	-	-	-	-
<u>Station Number 25</u>								
Broken Conductor	25.5	6.0	25.80	7.68	8.0	2.0	8.00	2.56
Broken Static	29.0	12.0	25.20	15.36	15.0	8.0	11.6	10.2
Dropped Insul.	11.2	4.0	10.24	5.12	.8	.4	0.64	-0.51

TABLE 14 (concl.)

Test	Peak Value				Final Value			
	δ_H	δ_V	H(oz)	V(oz)	δ_H	δ_V	H(oz)	V(oz)
Ice Conductor	11.4	2.4	11.76	3.07	-	-	-	-
Ice Static	26.0	16.0	18.40	20.48	-	-	-	-
Gallop Cond.(max p-p)	28.0	5.0	29.60	6.40	-	-	-	-
Gallop Static(max p-p)	33.0	7.0	34.00	8.96	-	-	-	-
<u>Station Number 26</u>								
Broken Conductor	14.0	2.3	14.96	2.94	3.0	0.9	11.09	1.15
Broken Static	9.0	1.9	9.28	2.43	3.0	0.75	3.00	0.96
Dropped Insul.	5.0	1.0	5.20	1.28	0	0	0	0
Ice Conductor	5.6	1.1	5.84	1.41	-	-	-	-
Ice Static	7.0	2.6	6.30	3.33	-	-	-	-
Gallop Cond.(max p-p)	10.0	2.0	10.40	2.56	-	-	-	-
Gallop Static(max p-p)	14.0	5.0	12.80	6.40	-	-	-	-
<u>Station Number 27</u>								
Broken Conductor	12.2	2.4	12.72	3.07	4.0	0.5	4.40	0.64
Broken Static	5.8	1.4	5.84	1.79	3.0	0.6	3.12	0.77
Dropped Insul.	-7.2	-1.4	-7.52	-1.79	-0.4	-0.3	-0.24	-0.38
Ice Conductor	5.6	1.0	5.92	1.28	-	-	-	-
Ice Static	6.2	0.9	6.72	1.15	-	-	-	-
Gallop Cond.(max p-p)	11.0	2.3	11.32	2.94	-	-	-	-
Gallop Static(max p-p)	10.0	2.0	10.40	2.56	-	-	-	-

by rank order I through III for each of the structures 23 through 27 as follows:

Summary of Maximum Dynamic Loading*

Structure Number	23	24	25	26	27
Load Rank					
I	(7)*	(2)	(2)	(7)	(5)
II	(2)	(1)	(1)	(4)	(3)
III	(6)	(5)	(5)	(2)	(6)

* Number indicates test type number, Table 13

Notice the shield wire dynamic loads account for the first, second and third rank of loading for all structures except structure 27. A number of specific comparisons will be made for clarity.

7.4 Comparison of Test Data

A. Comparison of structure 24 and structure 25 for long and short insulators under broken static wire test (test condition 2):

Comparison of Loading on Structure 24 and 25

	<u>Dynamic Load (oz)</u>	<u>Residual Load (oz)</u>	<u>I.F.*</u>
Structure Number 24			
Long	-21.00	-12.52	1.67
Short	-19.60	-12.45	1.57
Structure Number 25			
Long	20.4	12.52	1.63
Short	17.8	11.53	1.54

* I.F. = Impact Factor = Dynamic Load/Residual Load

The above indicates that the peak dynamic load is not very sensitive to insulator length nor to the flexibility difference between the two structures, the structure 25 being about 3 times as stiff as structure 24. The dynamic load is never greater than the reference load of 25 ounces (rigid structure broken shield wire) reaching only about 80 percent of that reference. The impact factor varies from a minimum of 1.54 to a maximum of 1.67. The residual load clearly shows the relaxation due to flexibility of both structures, being about 50 percent of the loading of the reference rigid structure. The minus sign indicates that the equivalent pole top load is toward structure number 22.

B. Comparison of structure 23 and structure 26 for long and short insulator under static wire test:

Comparison of Loading on Structure 23 and 26

	<u>Dynamic Load (oz)</u>	<u>Residual Load (oz)</u>	<u>I.F.</u>
Structure Number 23			
Long	-8.69	-4.41	1.97
Short	-6.69	-3.10	2.15
Structure Number 25			
Long	8.38	3.07	2.73
Short	4.28	1.56	2.74

The above indicates that the loads on these structures, each located one span length away from the broken span, are considerably less than the loads on structures adjacent to the break. The peak dynamic load on these structures is only about 1/3 of the reference load on a rigid structure. Of course, if all structures in the system were rigid then structures 23 and

26 would experience no load due to such a break in span 24 - 25. The fact that these structures are loaded probably accounts in large measure for the diminished loading of those structures adjacent to the broken span.

C. Comparison of structures 24 and structure 25 for long and short insulators under broken conductor test:

Comparison of Loading on Structure 24 and 25

	<u>Dynamic Load (oz)</u>	<u>Residual Load (oz)</u>	<u>I.F.</u>
Structure Number 24			
Long	-10.64	-2.32	4.58
Short	-10.07	-3.36	2.99
Structure Number 26			
Long	12.84	2.33	5.51
Short	12.57	4.16	3.02

These results show that the broken conductor loads the structure less than the broken static wire even though the initial tension is greater in the conductors. Also, while impact factors are high, the broken conductor loads appear to be diminished significantly due to insulator swing, the residual loads being less for long insulators. The residual loads are also low because the shield wire is intact, and provides support. However, the large impact factors focuses attention on the dynamic loads. The insulator length appears to have little effect on the peak dynamic loads. The peak dynamic loads appear greater on the stiffer structure (about 25 percent greater).

8. PRESENTATION OF FULL-SCALE DATA

The results of the model tests, which have been summarized in Table 13 and Table 14, may be used to obtain estimates of the full-scale loads on the actual structures under the same type of loading conditions. This transferral to the actual line is the major benefit of model testing. It delineates the primary structure loading conditions by rank order of importance, making possible the assessment of the adequacy of the design.

In making the transferral to the full-scale structures, it will be helpful to have in mind the loads to which the structures have been designed. The specified longitudinal design load for each of the structures 23 through 27 was 3,800 pounds(3.8 kips), the load to be applied at any one conductor, or at the overhead shield wire. While the application of this load to the shield wire support results in the maximum ground line moment for each structure, it was found earlier that maximum ground line moment was determined by transverse loads (see Table 1). Since the pole cross section is symmetrical, more than adequate strength has been provided to carry the 3.8 kip load at the shield wire.

8.1 Conversion to Full-Scale Loads

The model reactions H and V may be converted to an "equivalent pole top load" by the relation:

$$F_e = \frac{1}{1.3} \left(\frac{H}{5} + V \right)$$

But loads in the model at a geometric scale ratio of 1/50 convert to loads at full-scale in the ratio of $(50)^2 = 2,500$. After converting ounces to pounds, the full-scale equivalent to F_e becomes:

$$(F_e)_{F.S.} = 0.12 \left(\frac{H}{5} + V \right) \text{ kips} \quad (60)$$

The model test results in terms of H and V in ounces are tabulated in Tables 13 and 14. Rapid estimates of the Full-scale pole top load under any of the test conditions may be made by using Table 15 which has been constructed from equation 60.

8.2 Illustrative Example

Suppose it is desired to find full-scale loads on structure 24 due to ice unloading of the static wire. From Table 13 the peak dynamic reactions are:

$$H = -18.88 \text{ ounces}$$

$$V = -8.19 \text{ ounces}$$

By entering Table 15 with these values the full-scale equivalent pole top load is (by interpolation) $(F_e)_{F.S.} = 1.43$ kips. The corresponding ground line shear and ground line moment are:

$$GLS = 1.43 \text{ kip}$$

$$GLM = 1.43 \times \text{pole height}$$

$$= 1.43 \times 85 \times 12$$

$$= 1,458 \text{ in-kip}$$

Both shear and moment values are less than the design values.

TABLE 15

Table of $(F_e)_{F.S.}$

H/V	2	4	6	8	10	14	16	18
5	0.36	0.60	0.84	1.08	1.32	1.80	2.04	2.28
10	0.48	0.72	0.96	1.20	1.44	1.92	2.16	2.40
15	0.60	0.84	1.08	1.32	1.56	2.04	2.28	2.52
20	0.72	0.96	1.20	1.44	1.68	2.16	2.40	2.64
25	0.84	1.08	1.32	1.56	1.80	2.28	2.52	2.76
30	0.96	1.20	1.44	1.68	1.92	2.40	2.64	2.88
35	1.20	1.32	1.56	1.80	2.04	2.52	2.76	3.00

9. ANALYSIS OF RESULTS

All test results shown in Table 13 and 14 may be analyzed in terms of the equivalent pole top load F_e . This concept is convenient for comparative analysis because it not only refers all test data to a common base, but it implicitly includes the net result of interaction between shield wire and conductor. Also, by definition ground line shear (GLS) becomes F_e , and ground-line-moment (GLM) becomes $F_e \times$ pole height. Both of these quantities are of significance not only for the design of the base section of the pole shaft, but also for the design of the foundation.

The results of the broken shield wire tests are compared to broken conductor tests for both long and short insulator in Table 16.

9.1 Comparison of Dynamic to Residual Loads

Broken shield wire loads are higher than broken conductor loads at the two structures (24 and 25) adjacent to the break. Slightly higher dynamic loads are experienced at these structures for long insulator in both tests. Note that for the long insulator tests, a horizontal strut was used at station 25 to hold the conductor against lateral sway. The presence of this additional restraint is believed responsible for the somewhat higher loads. As to residual loads, the residuals are much higher for broken static when compared to broken conductor. This is consistent with the notion that the intact shield

TABLE 16
COMPARISON OF BROKEN SHIELD WIRE AND BROKEN CONDUCTOR
 F_e (kips)

	Broken Shield Wire		Broken Conductor	
	Peak	Residual	Peak	Residual
<u>Short Insulator</u>				
23	-1.044	-0.529	-0.360	-0.197
24	-2.352	-1.494	-1.208	-0.403
25	2.141	1.383	1.508	0.279
26	1.005	0.368	0.603	0.184
27	0.414	0.245	0.522	0.283
<u>Long Insulator</u>				
23	-0.802	-0.372	-0.517	-0.105
24	-2.520	-1.502	-1.276	-0.278
25	2.448	1.502	1.540	0.499
26	0.514	0.187	0.712	0.247
27	0.354	0.167	0.673	0.182

wire provides considerable influence in the longitudinal direction.

In terms of impact factors, the impact factors are much higher for broken conductor, again reflecting the reduced residuals for the latter.

Normally, long insulators are expected to provide load relief as compared to short insulators. This effect is seen only on those structures (23 and 27) which are remote from the strut supported insulator at structure 25, and then only on the residual loading of those remote structures.

The significance of these results is believed to be two-fold,

- 1) Maximum dynamic loadings occur for broken shield wire.
- 2) The maximum dynamic loadings are less than the load calculated on a rigid structure for a similar break (3.0 kips).

This latter point is further illustrated by comparing the loads on structure 24 and 25 where the effect of flexibility may be examined.

Comparison of the numerical values in Table 17 should also have in mind that the pole stiffness for structures 24 and 25 are respectively 58 #/inch and 147 #/inch. In other words, structure 25 is about 3 times as stiff. Also, when comparing long and short insulator, it should be remembered that a horizontal strut was used on long insulator tests at structure 25.

By inspection, the broken static wire produces comparable loads, on both structures, for long insulators, approximately 2.5 kips. The iced static unloading produced the highest load, 2.89 kips at structure 25. In general the stiffer structure produced the higher loads under all tests, as expected.

TABLE 17

Peak Dynamic Loads, F_e (kips)

	Structure 24		Structure 25	
	Short	Long	Short	Long
Broken Conductor	1.208	1.276	1.508	1.540
Broken Static	2.352	2.520	2.141	2.448
Drop Insulator	0.407	0.583	1.108	0.860
Ice Conductor	0.209	0.145	0.341	0.651
Ice Static	1.435	1.051	1.344	2.899
Gallop ing Conductor	0.289	0.311	0.597	1.478
Gallop ing Static	0.540	0.440	0.620	1.891

An unexpected, and somewhat surprising result is the high dynamic loading of the stiffer structure under galloping tests. The effect of the higher stiffness is pronounced, being almost twice the loading under galloping conductor on the stiffer structure for short insulators. The most startling comparison however, was for long insulators--again remembering that a horizontal strut is present at structure 25. Here, the combined effect of higher stiffness and horizontal strut restraint leads to galloping loads which are between 4 and 5 times higher on the stiffer structure. Further, the galloping static wire loads reach loads (peak-to-peak) which are about $2/3$ of the static wire initial tension. This is considered to be a significant finding, and may bring serious concern to designers. The primary concern being the possible fatigue damage to cross arms and cross arm supports. Further, the use of horizontal struts on angle structures may be open to question for the same reason.

9.2 Comparison of Dynamic Bending Moments to Design Bending Moments

The design ground line moment for each structure under longitudinal load is based on a specified pole top load of 3.8 kips. In no structure, does that determine the design. The design was actually determined by the lateral loads leading to the GLM shown in Table 1.

Thus, from the standpoint of strength, the ground line strength of these structures far exceeds any of the dynamic ground line moments. However, the upper sections of the pole shaft were designed according to 3.8 kips load applied to the top, and the arms were designed to a 4.0 kip

load applied vertically (heavy ice), while the longitudinal design of the arm was 3.8 kips. Again referring to Table 1, none of these design loads are exceeded, but considering that the galloping loads are as high as 1.89 kips, there is a reasonable doubt concerning the fatigue life of the arms.

9.3 Applications to Design of Poles and Foundations

These model test results have demonstrated that it is possible to simulate a number of dynamic loading conditions on a scale model of an actual transmission line system, taking into account a large number of variables, and producing results having direct interpretation at full scale. Ground line shears and ground line moments have a direct meaning for foundation design, and to pole design-particularly the base section.

The method appears to be particularly useful in identifying any potential "trouble areas" in an existing design such as the use of horizontal struts on light angle structures, and/or adequacy of fatigue life.

10. CONCLUSIONS AND RECOMMENDATIONS

The principal conclusions of this investigation are the following:

- 1) A method for rapidly calculating the flexibility (stiffness) of steel pole transmission structures has been developed. The method relies on computer print-out data supplied by pole manufactures at the time of bid evaluation.
- 2) Model structures were designed and fabricated to match the full-scale stiffness influence coefficients to a scale ratio of 50:1. A model line consisting of 11 spans on a scale ratio of 50:1, was designed and erected. It consisted of an overhead shield wire and one (upper phase) conductor under static tension of NESC heavy ice equivalent.
- 3) Instrumentation was developed for the measurement of reactions on the model under seven different dynamic loading conditions at each of five structure locations. Several independent methods of calibration were used to insure reliable results.
- 4) Test results were reduced to a single parameter to characterize the dynamic and residual load response. The equivalent pole top load concept compares full-scale loads under each of the seven test conditions and at each of five structure locations.
- 5) Results of broken shield wire tests show that structure flexibility reduces both the dynamic load and the residual load below the load calculated for a rigid structure.

6) The maximum loads occur for broken static wire on the most flexible structure, and for ice unload of static wire on the most stiff structure. The latter result is subject to the further restraint of a horizontal strut at a light angle.

7) Unexpectedly high loads were found on the stiffer structure with long insulators (including the horizontal strut) for galloping conductor and galloping shield wire tests. These relatively high loads (about 63 percent of initial shield wire tension) are considered a possible cause for concern about the fatigue life of the upper conductor arm, and/or upper sections of the pole shaft.

10.1 Recommendations

- 1) Although damping was not considered a part of the investigation, inspection of the test records reveals the severe loading unbalance, such as conductor or shield wire breaks, are damped more heavily than the dropped insulator or galloping conditions. This suggests that the latter conditions, involving as they do, coupling throughout the system may be subject to greater influence by damping. Studies which include damping are recommended.
- 2) The small sags and short spans in the investigated line may limit the conclusions because of the relatively small slack in these spans. In longer spans the catenary stiffness will be substantially less. A much more detailed study of the effects of initial slack in longer spans is recommended.
- 3) The full scale line structures represented by this investigation were all designed by critical lateral loading. Other lines whose structures

are designed primarily by longitudinal loading should also be studied.

4) A horizontal strut on a light angle structure, appears to amplify the dynamic loading--particularly for the galloping tests and ice unloading. This effect should be carefully examined.

5) While the investigation considered the steel pole type of structure which characteristically employs foundation rigidity, other structures having low foundation rigidity such as guyed V and H frame should be studied on a model scale.

APPENDIX

A general idea of the mechanism by which the shield wire and conductor interact is provided by a qualitative study of residual values of the sag in various spans following either a conductor break or a shield wire break.

Following such breaks in the model in span 24 -25, measurements were made of the mid-span sag changes from normal (no breaks). These data were analyzed and expressed in terms of full-scale tension. See Table 18.

TABLE 18

Full-Scale Tension (kips) Calculated from Sag Measurements

	Broken Shield Wire		Broken Conductor	
	Shield Wire	Conductor	Shield Wire	Conductor
Span				
22-23	2.31	5.0	2.85	2.80
23-24	1.69	4.79	2.92	2.73
24-25	x	6.29	5.17	x
25-26	2.23	5.18	3.43	2.23
26-27	2.35	5.75	3.12	3.27

Original Tension: Shield Wire - 3.0 kip, Conductor - 5.0 kip

Note: The accuracy of the sag measurements is not considered sufficiently precise to establish the loading on the structures by this method.

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