

Temperature Effects on Tall Steel Framed Buildings

Part 2—Structural Analysis

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RECENT TRENDS in architectural design have led to the construction of multistory structures with the exterior columns partially or fully exposed. Such an arrangement provides a pleasing aesthetic effect with strong vertical lines. However, this arrangement subjects the exposed columns to severe temperature variations, whereas the interior columns remain in a controlled environment. These variations induce changes of length of the exterior columns which induce forces throughout the structure. The magnitude of these forces depends on many things, but of primary importance is the height of the structure. For low structures, the effects are small; however, for tall buildings they may be significant.

The response of structural members to temperature variations is well known, but little work has been done on the analysis and design of tall steel buildings in which temperature changes occur in the exposed columns. Khan and Fintel developed a procedure for the analysis of temperature effects in reinforced concrete structures.^{1, 4} Their procedure is an approximate iteration process which assumes rigid beam-to-column connections. This assumption is valid for reinforced concrete structures; however, steel structures may have rigid or semirigid connections.

There are several schemes of complete structural analysis available to the designer that can be used for the temperature problem. The Structural Design Language (STRUDL) of Integrated Civil Engineering System (ICES)³ is an example of such a scheme. This problem-oriented program provides a stiffness method of analysis for the complete frame. However, at present, semirigid joints and complicated temperature gradients cannot be handled. STRUDL is used for comparison in this study; however, the cost prohibits its use for the parameter study.

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SCOPE OF STUDY

The primary objective of this study is to develop a method of analysis for determining the member forces and joint displacements that result in steel frames from specified temperature changes in the exterior columns. For this study, consideration is limited to plane frames.

Several limited studies are conducted on a few typical multistory structures. The intent is to demonstrate the method of analysis and to ascertain what the important parameters are in causing and limiting the effects of temperature changes. The studies made include:

1. Influence of number of stories on temperature effects
2. Influence of beam stiffness on temperature effects
3. Influence of connection stiffness on temperature effects (results not reported here)
4. Influence of temperature gradient on temperature effects

The above studies are conducted using two typical multistory steel structures. These structures are described later.

MATHEMATICAL MODELS AND LOADING CONDITIONS

Mathematical Models—The temperatures changes that occur in the exposed columns of multistory structures induce deformations and forces throughout the structure. For multiple-bay structures, these effects die out quite rapidly in successively adjoining inner bays.

The complete frame could be used in studying each loading condition that is induced by temperature change. However, as a part of this study, an attempt has been made to determine whether some simplified model can be used for determining critical response values. The models used in various parts of this study are briefly described in the following paragraphs.

Single-Bay Model—Since the maximum effects occur in the outer bays, a model structure is suggested that considers the outer bays as being attached to a rigid central

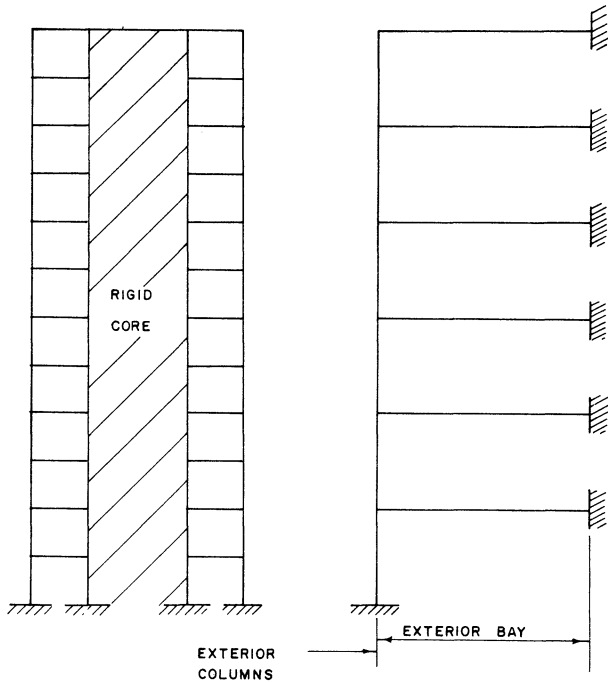


Fig. 1. Single-bay model

core. Such a scheme is shown in Fig. 1. One obvious shortcoming of this arrangement is that only forces and displacements in the exterior columns and exterior-bay beams are determined. This in itself is not a serious shortcoming, since the maximum effects occur in these members. However, the assumed fixity at the inner edge of the outer bay produces a significantly greater stiffness of this bay with respect to temperature-induced loading than that of the real structure. In general, this false fixity leads to smaller displacements and larger member forces than would actually occur in the real structure.

Two-Bay Model—The shortcomings of the one-bay model can be partially overcome by assuming a two-bay structure to be attached to a rigid core. This arrangement is shown in Fig. 2. In this case, the forces and displacements are determined in two rows of columns and two bays of beams. The exterior-column and exterior-bay values, which are the most critical, are closer to the true values because the restraining effect of the inner bay on the outer bay is more realistic than the fixity used for the single-bay model. The values determined for the second row of columns and the interior bay are less reliable because of the fixity at the rigid core.

STRUDL Model—In this study, STRUDL is used to treat the complete structure. The results so obtained are then taken as the correct values so that the results of the two approximations enumerated earlier can be evaluated by comparison.

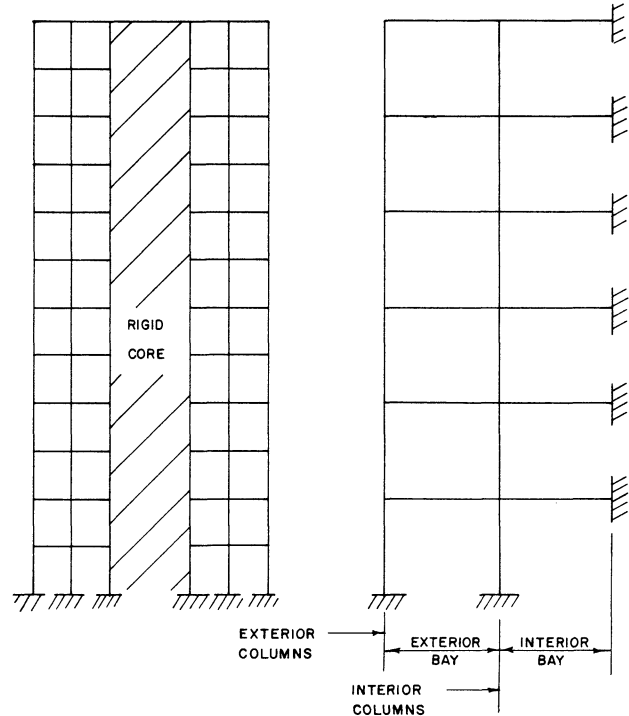


Fig. 2. Two-bay model

Loading—The loading on the frames under consideration is induced by temperature changes in the exterior columns. The temperature change may have one of three possible gradients across the column section: zero slope, constant slope, or variable slope. These three cases are shown in Fig. 3.

METHOD OF ANALYSIS

A brief summary of the method of analysis follows. The reader is referred to a more detailed presentation by the authors,⁸ in which the equations are developed in detail.

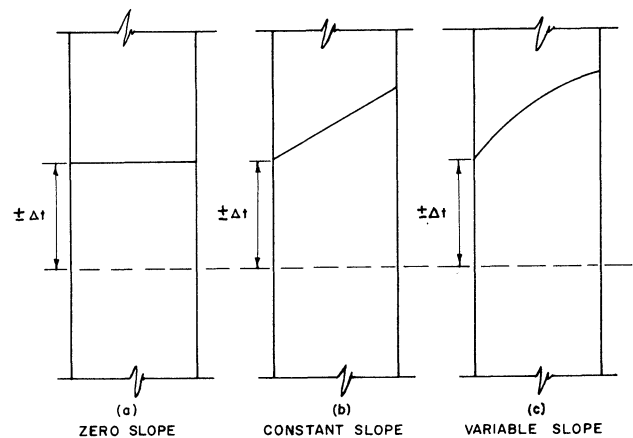


Fig. 3. Temperature change across column section

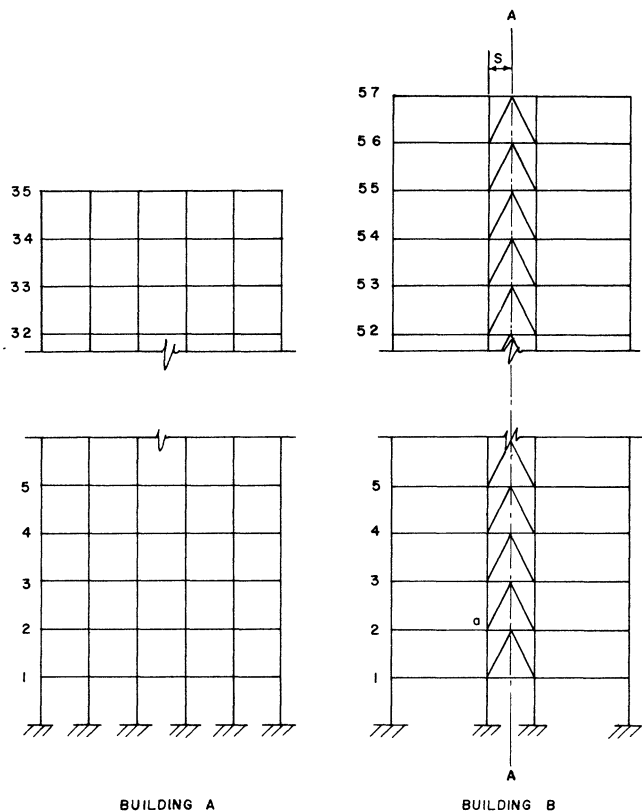


Fig. 4. Test structures

Principle of Minimum Potential Energy—The method of analysis used is based on the principle of minimum potential energy. For equilibrium, the change in the total potential energy corresponding to any virtual displacement that does not violate the constraints is equal to zero.

The assumption is made that temperature loading can be superimposed on the other conditions of loading. Thus, for a system having n degrees of freedom, the total potential energy, U can be expressed in terms of the unknown displacements $\lambda_1, \lambda_2, \dots, \lambda_n$ corresponding to the temperature loading and the temperature change, Δt . A variation of this potential energy corresponding to some general set of virtual displacements can be expressed as

$$\delta U = \frac{\partial U}{\partial \lambda_1} \cdot \delta \lambda_1 + \frac{\partial U}{\partial \lambda_2} \cdot \delta \lambda_2 + \dots + \frac{\partial U}{\partial \lambda_n} \cdot \delta \lambda_n \quad (1)$$

and since for equilibrium, the change in potential energy must vanish for any virtual displacement, it follows that

$$\partial U / \partial \lambda_i = 0 \quad (i = 1, \dots, n) \quad (2)$$

This leads to n linear algebraic equations which are the governing equations of equilibrium.

Equations of Equilibrium—The governing equations of equilibrium, when cast in matrix form, are:

$$[K]_{n \times n} \{ \lambda \}_{n \times 1} = \{ P_t \}_{n \times 1} \quad (3)$$

where $[K]$ = the stiffness matrix which is a function of the properties of the structure, $\{ \lambda \}$ = the vector of displacements resulting from the imposed temperature changes, and $\{ P_t \}$ = the vector of equivalent temperature loads which is a function of the structural properties and of the temperature change. The precise character of the stiffness matrix depends on the nature of the assumed structure. In the authors' detailed work,⁸ the stiffness matrix reflected the inclusion of rigid or semirigid beam-to-column connections. However, the results reported here only deal with cases where rigid connections are assumed.

Displacements and Member Forces—The solution of Eq. (3) yields the joint displacements corresponding to the specified temperature loading. Using these displacements and the specified temperature changes, all of the member forces throughout the structure may be determined.

COMPUTATIONAL STUDIES: OBSERVATIONS AND INTERPRETATIONS

In this section, the method of analysis outlined above is applied to some specific cases and the results are presented and interpreted.

Description of Buildings Studied—Two test structures are used in the computational studies. The first, identified as Building A, is a 35-story structure with 5 bays. The second structure, Building B, is a 57-story structure with 3 bays. These structures are shown in Fig. 4. In Building B, the inner bay constitutes a braced core. To make this structure conform to the two-bay approximation, the portion of the structure left of line A-A is taken with fixity being assumed at this line. In the resulting two-bay frame, the properties used for the beams of the interior bay are determined in a manner that makes them flexurally equivalent to the truss-type arrangement of the actual structure. For instance, at point *a* of Building B in Fig. 4, the equivalent interior-bay beam is taken to have a span S and a rotational stiffness equal to the sum of the rotational stiffness of the two members framing into point *a* from the righthand side. For stiffness calculations, both of these members are assumed to be fixed at line A-A.

Comparison of Models—As mentioned earlier, several different mathematical models are used in this study to represent the actual structure. In this section, the results obtained by using the different models are compared.

This comparison is based on studies involving Building A. A temperature change of -45° (uniform across the cross section) is assumed for all the exterior columns.

In the discussion which follows, STRUDL results represent the correct values corresponding to a complete frame analysis. Other results are identified by the mathematical model employed. Reference to exterior columns and bays, and interior columns and bays is consistent with the identification of Figs. 1 and 2.

Figure 5 shows the vertical joint displacements at various story levels as computed by using the various models. The single-bay model yields exterior vertical joint displacements that are about 10 percent less than the correct values at all story levels. This reflects the fact that the single-bay model has greater vertical stiffness relating exterior vertical joint displacement to temperature induced loading than has the actual structure. This increased stiffness is because of the assumed fixity at the connection to the rigid core. The exterior displacements for the double-bay model are very nearly equal to the correct values. They are about 2 percent less than the correct values, because this model, too, has a greater stiffness than the true structure. Again, this increased stiffness stems from the assumed fixity at the core; however, since this condition is farther removed from the exterior columns, its effect is less significant.

Interior vertical joint displacements are available only for the STRUDL analysis and the two-bay structure. The two-bay values are about 20 percent less than the correct values. This discrepancy is again attributable to the difference in stiffness between the two models—the two-bay model being the stiffer of the two with regard to vertical interior displacements in response to temperature-induced loading in the exterior columns. The rather large discrepancy here is not too significant, however, because these displacements are small compared to the exterior displacements.

Actually, the absolute values of exterior and interior vertical displacements are not too important. Instead, it is the relative displacement between the exterior and interior joints that is of prime importance. It is this relative displacement that contributes to the magnitude of the forces induced in the exterior bay. Also, excessive relative displacements may impair a structure's serviceability characteristics. Figure 5 shows that the single-bay model yields relative displacements for the exterior bay that are about 12 percent greater than the true values, while the two-bay model overestimates the correct values by about 3 percent.

Figure 6 shows the beam moments in the beams of the exterior and interior bay that are caused by the temperature-induced loading. The moment plotted in these figures is the left-end moment at each story level. The relationships between the values for the different models for the exterior bay reflects the same pattern as was ob-

served for the relative vertical displacements for the exterior bay. Of course, these moments involve the end rotations as well as the relative displacements; however, the relative displacement dominates for temperature-induced loading.

For the interior bay, the discrepancy between the results for the two-bay model and the correct values is considerably larger than that for the exterior bay. Here, the two-bay values run about 50 percent greater than the true values. This again reflects the fact that the two-bay model does not give good results for the interior bay because of the assumed fixity at the core. However, these interior bay forces are considerably smaller than those induced in the exterior bay.

Figure 7 shows the axial loads in the columns at the various story levels for the different models. Axial loads in the exterior columns would be greatest for infinitely stiff beams which would permit no vertical displacements of the beam-to-column connection points. If the columns were completely free to elongate, the axial loads would be zero. Thus, the model which produces the smallest exterior vertical displacements (single-bay model) yields the largest axial load. In this case, these loads are about 25 percent higher than the true values. The double-bay model gives the next highest axial loads; however, these are only 2 percent greater than the correct values.

Loads in the interior columns are induced by end deformations. Thus, the largest axial load for interior columns is associated with the model which produces the largest vertical interior displacements (STRUDL). The two-bay model gives values about 25 percent lower than the actual values.

Location of Maximum Effects—The comparisons made in the previous section enable us to identify the locations of the maximum effects.

Figure 5 clearly shows that the maximum joint displacements increase with story location. This is true for both interior and exterior column lines. Interior displacements are only about 20 percent of the exterior displacements, thus indicating rapid dissipation of effects.

Moments in the exterior bay tend to increase with increasing story location, as shown in Fig. 6. This increase, however, is not steady. Even though end displacements and end rotations for the beams increase quite uniformly with story position, changes in beam sections make the moment vs. story height curve irregular. The largest moment occurs in the beam that is one level below the roof. Even though the displacements and rotations for this beam are slightly less than those at the top level, the end restraint is greater. Thus, the beam has greater shear and moment. Figure 6 shows how quickly the moments dissipate as we progress toward inner bays. Peak moments in the first interior bay are only about 30 percent of the exterior values.

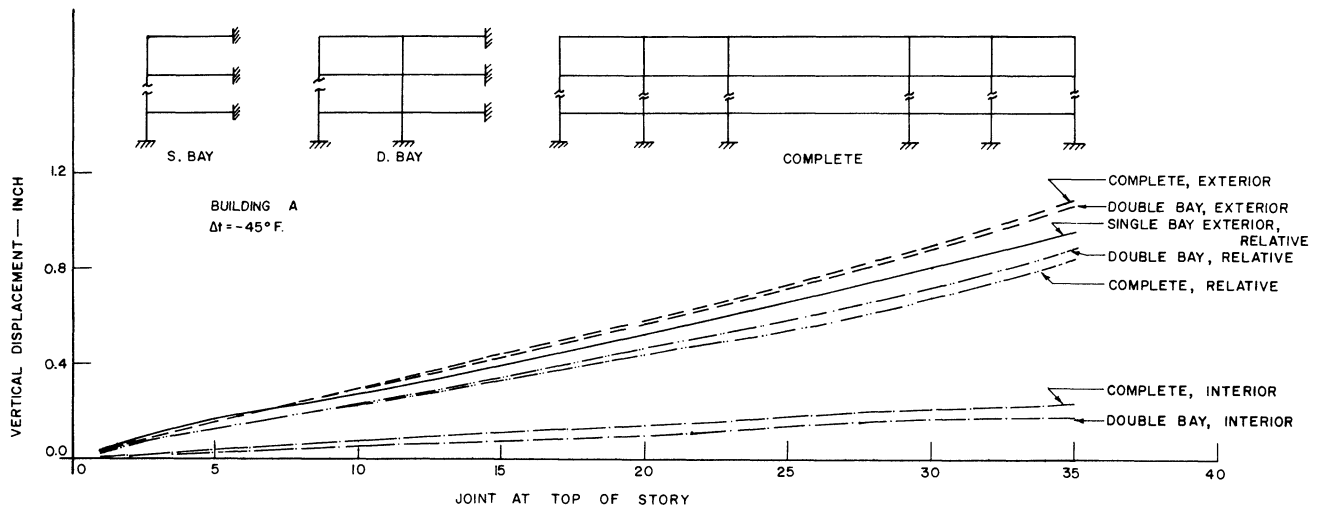


Fig. 5. Vertical displacement vs story location

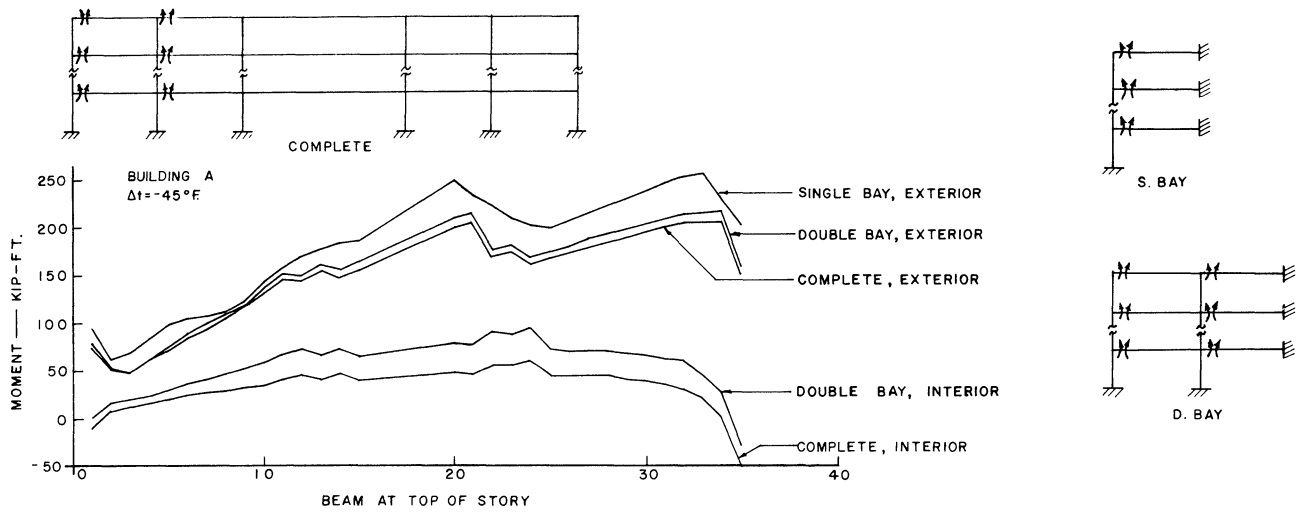


Fig. 6. Beam moment vs story location

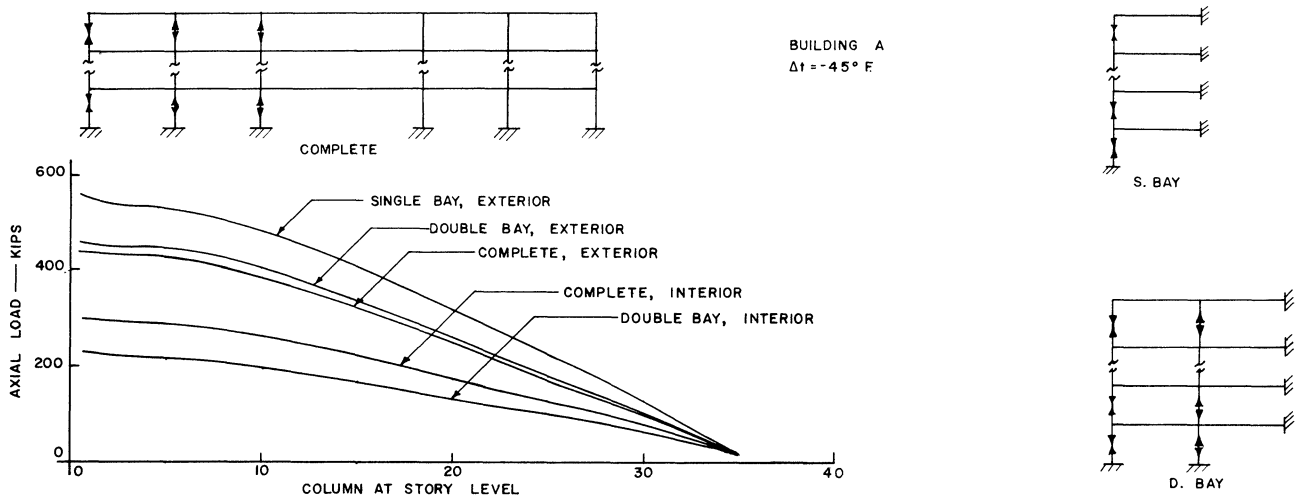


Fig. 7. Column axial load vs story location

Axial loads are greatest in the first story columns. These loads diminish steadily as we progress up the structure. Again, the effects reduce as we advance inward. However, in this case, the dissipation is less pronounced. The first interior columns have forces equal to slightly more than one-half of the exterior column forces.

For the two-bay model, the location of the maximum effects are shown in Fig. 8. Clearly, the exterior columns and exterior bay beams are the members most seriously affected by temperature changes in the exterior columns. The effects dissipate rapidly away from the exterior of the structure. Any model that yields good predictions of displacements and forces in the exterior region of the structure would be helpful for design purposes.

Influence of Number of Stories—The purpose of this section is to examine the influence of the number of stories on the resulting temperature effects.

Attention is focused on the displacements and member forces in the exterior portion (exterior column line and exterior bay) of the structure. This procedure is in accord with the findings of the previous section, where the response is shown to be most critical in this area. Only the results of the two-bay model study are given, since this model has been shown to give good results. This is especially true in the exterior portion of the frame, where the two-bay model gives results within a small percentage of the correct results.

For this study, both Building A and Building B are used. For both buildings, a structure with a specific number of stories is formed by isolating the topmost part of the structure with the desired number of stories. The number of stories is thus varied by successively isolating a different number of stories.

The curves plotted are for a change of temperature of -10°F (uniform across the cross-section). Since the structural response is linear with respect to temperature change, the effects for other temperature changes can be easily determined.

Vertical Displacements of Beam-to-Column Connection Points—Figure 9 shows the variation in the relative vertical displacement of the topmost beam as the number of stories is varied. The ordinate has been normalized with respect to the free shortening that the exterior column would undergo if it were not restrained by the rest of the structure.

The general behavior is similar for the two structures studied. As expected, the displacements are reduced when the composite nature of the floor system is accounted for. For both composite and non-composite structures, increasing the number of stories leads to more beams and greater vertical stiffness with respect to temperature-induced loading. Thus, with increasing number of

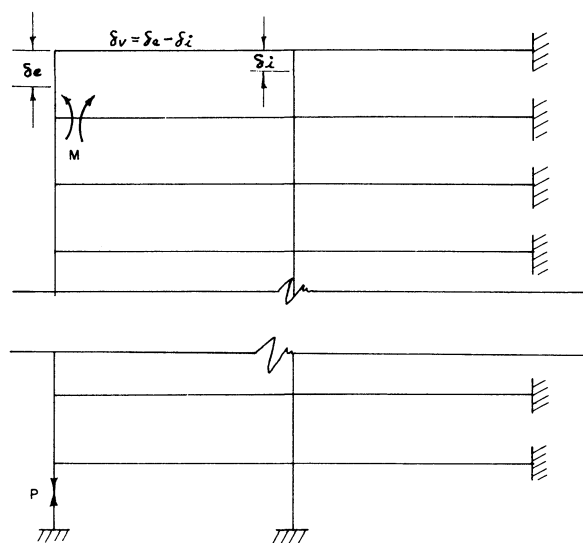


Fig. 8. Location of maximum effects

stories, a decreasing percentage of the free displacement occurs, even though the absolute value of displacement is increasing at a decreasing rate.

It should be noted that other factors which increase the overall stiffness of the structure, such as partitions, would further decrease the displacement.

Bending Moments—As previously demonstrated, the maximum beam moment occurs at the left end of the exterior beam which is one level below the top of the structure. Figure 10 shows how this moment varies as the number of stories is altered. The ordinate has been nondimensionalized by the yield moment for the beam under consideration.

Both buildings reflect the same general behavior, with the moment increasing at a decreasing rate as the number of stories increases. This is consistent with the displacement pattern already discussed.

The plots of Fig. 10 indicate the portion of the total yield capacity being exhausted by temperature-induced moments. The discrepancy between the two buildings can be explained by realizing that existing structures are being analyzed. Building A was designed for a more moderate climate than was Building B. Thus, it seems reasonable that whatever scheme was used to take into account the effects of temperature change, Building A was considered to have smaller temperature differences than Building B. Thus, the moments induced by a given temperature change constitute a greater percentage of the total yield moments in the case of Building A than in the case of Building B.

Figure 10 shows also the response when composite action is estimated. It is seen that composite action has a relatively small effect on the results. Even though the displacements are reduced by composite action (as in-

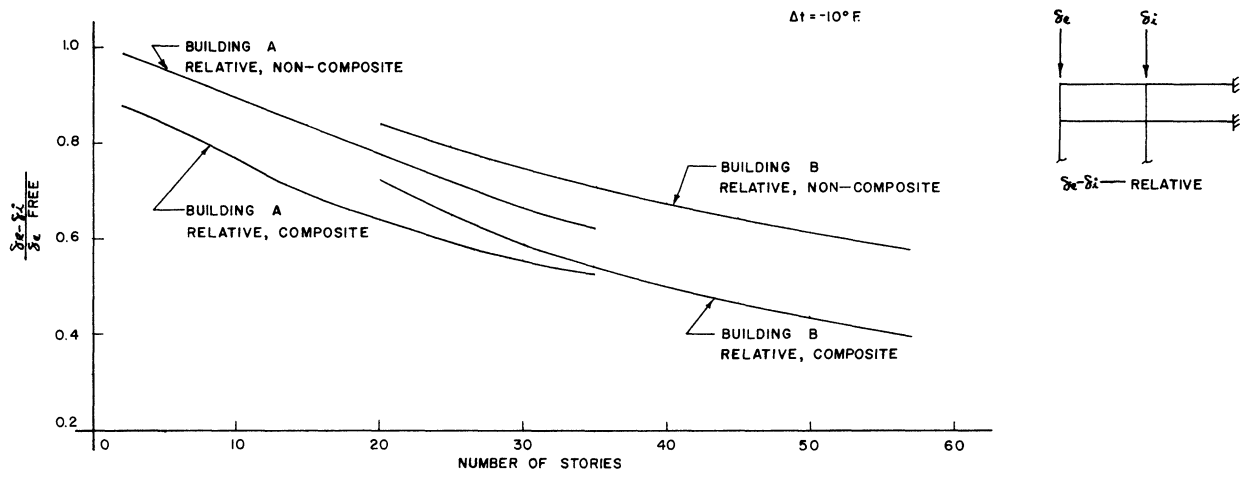


Fig. 9. Normalized maximum relative vertical displacement vs number of stories

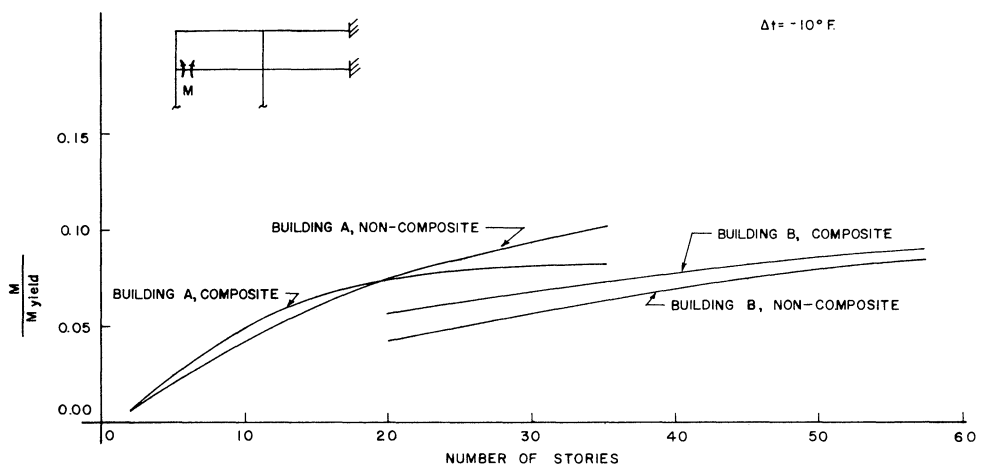


Fig. 10. Normalized maximum beam moment vs number of stories

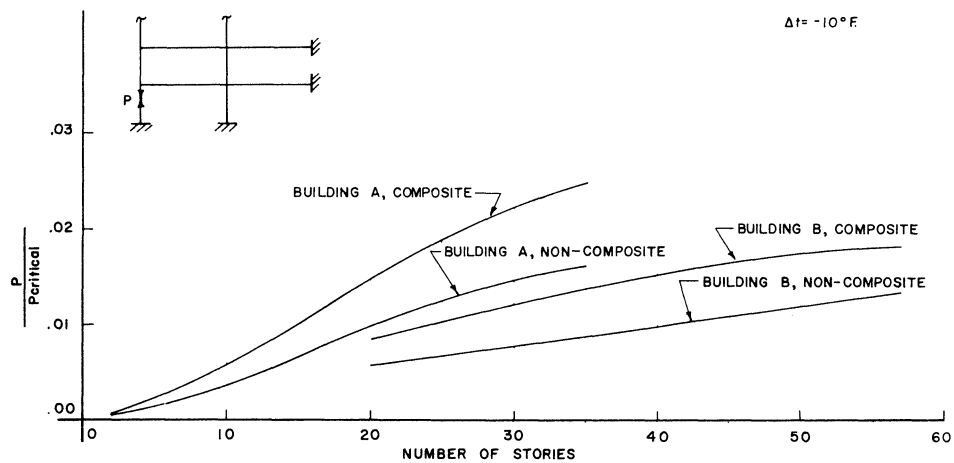


Fig. 11. Normalized maximum column axial load vs number of stories

licated earlier), the members, through their increased stiffness, still develop essentially the same moments as in the case of the noncomposite structures.

Axial Loads—As indicated earlier, the column that is most severely stressed from temperature-induced loading is the exterior column for the first story.

Figure 11 shows how the ratio of the maximum temperature-induced axial load to the corresponding critical buckling load varies as the number of stories is increased. It is seen that both buildings reflect the same general behavior. As pointed out earlier, the greater the restraint against free elongation of the column, the greater is the force induced by temperature change. The upper end of the first-story column is restrained more and more as additional floors are added. This accounts for the increase in axial load at a decreasing rate as the number of stories is increased. It is also seen that the stiffening action which takes place when composite action is considered sharply increases the axial loads. Other factors which act to stiffen the structure would also tend to increase the axial load.

This plot also shows that axial loads introduced for a given temperature change constitute a greater percentage of the allowable axial loads in the case of Building A than in the case of Building B.

Influence of Beam Stiffness—The movements induced by temperature changes in the exterior columns are dependent upon the overall vertical stiffness of the structure with respect to the temperature induced loads. One factor in determining this stiffness is the stiffness of the individual beams.

Attention is focused on the response quantities of the exterior column line and exterior bay. This is consistent with the observed locations of maximum effects. Both Buildings A and B are treated using the two-bay model.

Figure 12 shows the variation in various response functions as the beam stiffness is varied. In this figure, the abscissa is entitled "beam stiffness factor". This term represents the factor by which all the actual beam moments of inertia for a given structure have been multiplied. The ordinate is the response function, nondimensionalized by the value of the corresponding response function for the actual structure (beam stiffness factor = 1); thus, the resulting ordinate for any curve reflects the ratio of the given response function for any beam stiffness factor to the value of the corresponding function for the actual structure. All curves in Fig. 12 are for a uniform temperature change of -10°F .

Vertical Displacements of Beam-to-Column Connection Points—The variation in the relative vertical displacements for the topmost exterior beam, as the beam stiffness factor varies, is shown in Fig. 12. Both structures exhibit the same general behavior, in which the displacements drop off sharply at first and then more slowly as the beam stiffness increases.

Beam Moments—As shown earlier, the critical moment occurs at the exterior end of the floor which is one level below the roof level.

The variation in this moment as the beam stiffnesses are increased is shown in Fig. 12 for the two buildings under study. For both buildings, the displacements are reduced by increasing the beam stiffness, and the moments induced in the beams increase.

Axial Loads—The largest axial load is registered in the first-story exterior column, as pointed out earlier.

Figure 12 shows how this axial load changes with changing beam stiffnesses for the two structures under consideration. The greater the restraint with respect to vertical movement at the top of this first-story column, the greater is the axial load induced by temperature loading. Since increasing the beam stiffnesses tends to increase this restraint, the axial load increases steadily as the beam stiffness is increased. Both structures show the same general trend.

Influence of Temperature Gradient on Temperature Effects

—Thus far, only cases in which the temperature change is constant across the cross section have been considered. This condition would only exist for fully exposed columns. Such an arrangement is not the most common. In most cases the columns are partially exposed in such a way that there is a temperature gradient of the types indicated in Figs. 3(b) and 3(c).

This kind of temperature variation gives rise to equivalent temperature moments in addition to the equivalent temperature loads already discussed. These moments are part of the load vector in Eq. (3). It is convenient to think of the average temperature change over the cross section as inducing the equivalent axial loads and the remainder of the temperature variation as inducing the equivalent moments.

Since the case of uniform temperature change has already been studied, this section is devoted to studying the effects of temperature gradient. The effect is measured by determining the relationship between a particular response quantity caused by a given temperature gradient and the corresponding response quantity caused by the average (uniform) temperature change. In this way, it can be determined just how significant various temperature gradients are at different levels of average temperature change.

In this section, only gradients of constant slope (Fig. 3b) are considered. Building A is used with the two-bay approximation. Gradients with extreme variations ranging from 0° to 50°F between interior and exterior faces are assumed to act with average temperature changes of 30° , 45° , 60° and 75°F . All combinations of the above are presented and others can be determined from interpolation.

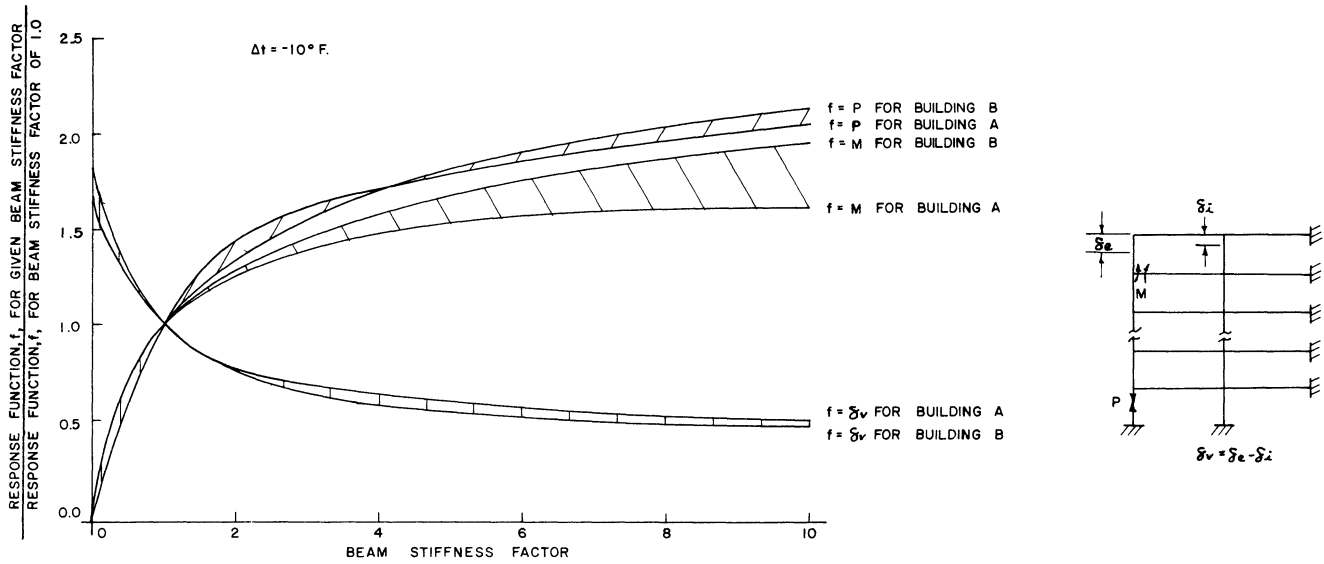


Fig. 12. Normalized maximum response function, f , vs beam stiffness factor

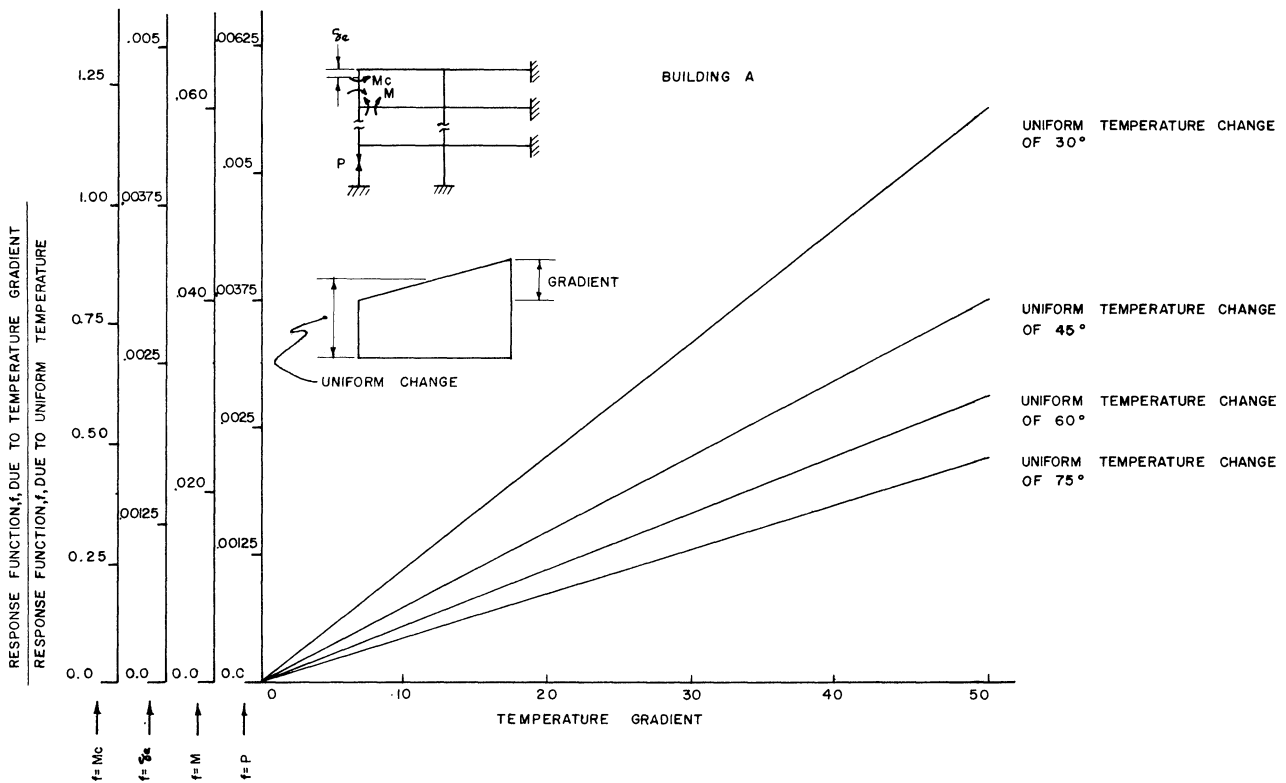


Fig. 13. Effect of temperature gradient on maximum response functions

In Fig. 13, for any combination of uniform temperature change and gradient cited above, the ratio between a given response quantity induced by gradient to the corresponding quantity induced by uniform temperature change is given by the appropriate ordinate.

Response Quantities of Interest—The response quantities that we have been examining thus far (relative vertical displacement in the topmost beam, exterior-end moment in the exterior-bay beam located one level below the roof, and axial load in the first story exterior column) have

been those that are critical for uniform temperature change. Each of these response quantities is related to the vertical movement of the exterior beam-to-column connection points. This movement is greatest for uniform temperature changes which impose equivalent temperature loads that are axial loads in the exterior columns. However, temperature gradient induces equivalent temperature loads that are moments at certain beam-to-column connection points along the exterior column line. These moments do not cause much vertical movement of the building. This is shown in Fig. 13, where it is seen that the ratio of response functions due to gradient to those due to uniform temperature are small for those functions related to vertical movement of the structure (δ_e , M and P). This is true even when the gradient is large and the uniform temperature change is small.

However, the temperature-induced moments along the exterior column associated with temperature gradient do cause large moments in the exterior column. In Fig. 13, when the moment at the base of the top-story column is taken as the response function, the ratio of gradient effect to uniform temperature effect is quite large.

Typical Gradients—Studies made by McLaughlin,⁶ on semiexposed steel building columns with fireproofing and metal covers, indicate a typical temperature gradient of 5°F between the two faces of the column. Figure 13 indicates that the effect that this gradient has on all functions except the column moments is insignificant, regardless of the accompanying uniform temperature change. In the case of column moment, the ratio of gradient effect to uniform-temperature effect is approximately 0.10 for a uniform change of 30°F.

Architects are beginning to display a tendency toward employing columns that are partially exposed without any fireproofing or cover of any kind. McLaughlin's studies⁶ indicate that gradients may be as high as 40°F for this arrangement for normal temperature variations. Such a variation would be accompanied by a rather large uniform temperature change. For a gradient of 40°F and a uniform temperature change of 75°F, Fig. 13 again reveals that only the column moment is significantly affected by the gradient effect where the ordinate is approximately 0.40.

COMPARISON OF COMPUTED AND OBSERVED VALUES

Field observations have been made on Building B, in which certain displacement quantities were observed. These observations were made by consulting engineers Edwards and Hjorth⁷ and were reported along with similar measurements for several other buildings. The relative displacement of the exterior beam at the roof level of Building B was observed to be $-\frac{3}{4}$ -in. for a temperature change of -45°F .

The free displacement for the exterior columns of this building that would be associated with a 10°F temperature change is 0.60 in. From Fig. 9, the ratio of the relative displacement to the free displacement $[(\delta_e - \delta_i)/\delta_{free}]$ for the full 57-story noncomposite structure subjected to a temperature change of 10°F is 0.55. Thus, the relative displacement would be $0.55 \times 0.60 = 0.33$ in. For a temperature change of 45°F , the relative displacement would be $0.33 \times 45/10 = 1.48$ in. This is nearly double the observed displacement.

If the composite nature of the floor system is considered, Fig. 9 indicates that the ratio of relative displacement to free displacement for a 10°F temperature change is 0.37. Thus, for 45°F , the relative displacement is $0.37 \times 0.60 \times 45/10 = 1.02$ in. This value is still about 33 percent above the observed value of $\frac{3}{4}$ -in. The remainder of the discrepancy can be explained in two ways. First, in addition to composite action, the vertical stiffness of the actual structure is further increased by the effects of partitions or any other appurtenances which contribute to vertical stiffness. Second, even though temperature observations indicated a temperature change of -45°F when the $\frac{3}{4}$ -in. displacement was observed, this does not ensure that the steel columns were actually subjected to this temperature change. In all likelihood, it was something less than the full -45°F because the columns are covered with concrete, insulation material, and cladding.

CONCLUSIONS

The studies made as a part of this investigation lead to the following conclusions:

1. A complete frame analysis using STRUDL can generally be used to determine the important response quantities that result from temperature-induced loading. This is not possible when semirigid connections are employed or when complicated temperature variations are specified. Such a complete analysis is expensive and is not available to every designer. In this study, a two-bay model in which the two exterior bays are assumed to be connected to a rigid core is found to give excellent results for the critical response values in the exterior column line and exterior bay. The same model gives somewhat less reliable results for the first interior column line and bay; however, these values are much smaller than the exterior values and are less important.

2. The maximum vertical displacements occur at the roof level of the structure. The greatest absolute displacement is at the exterior column line, whereas the maximum relative end displacement occurs in the exterior bay of the roof beam.

The exterior bay beam that is one level below the roof is subjected to the greatest moment. The largest axial load occurs in the first floor exterior column.

It should be noted that all of the above quantities, which represent the maximum effects, occur in the exterior column line or exterior bay. Since excellent results for the two-bay model are obtained in this region of the structure, it indicates the adequacy of this model in determining the critical response quantities.

3. There appears to be a strong correlation between the critical response quantities and the number of stories. All response quantities exhibit increases as the number of stories increases — displacements and moments at a decreasing rate and column axial load at an increasing rate. The number of stories, however, is not the only factor in determining whether or not a response quantity is acceptable. Even though the number of stories may indicate the magnitude of the displacements, the span over which these displacements occur must be considered.

The structures studied in this investigation show the same general behavior as the number of stories is varied. Nondimensionalized plots, such as Figs. 9, 10, and 11, designed to emphasize the common behavior between different structures, may be formalized for design purposes.

4. Beam stiffnesses play an important role in contributing to the overall vertical stiffness of a building. Increasing beam stiffnesses reduce the critical displacements induced by temperature loads while increasing the important member forces. Thus, it follows that when the composite nature of the floor system is considered, the displacements are reduced and the member forces are increased.

Nondimensionalized curves such as Fig. 12, relating temperature response values to changes in beam stiffness, show excellent agreement between the two structures studied. Further study here should lead to good design data.

5. Any given temperature variation across a column can be considered to be composed of two parts — a uni-

form temperature change and a superimposed departure from this uniform pattern. This latter part is referred to as the temperature gradient.

The effects of the temperature gradient on the important response quantities related to vertical movement of the building are only a very small percentage of the effects caused by the associated uniform temperature change. This is true even for gradients as large as 50°F between the two column faces coupled with a uniform temperature change of only 30°F.

There are certain cases where the effect of the gradient is greater than the effect of uniform temperature change. This can be true for the column moments where temperature gradients induce large column moments.

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