Temperature Effects on Tall Steel Framed Buildings

Part 3—Design Considerations

Fazlur R. Khan and Anthony F. Nassetta

SINCE THE beginning of high rise construction in steel late in the 1800's up to about 1940, the structural skeleton was hidden behind the exterior facade in almost all cases. Typically, such construction used stone or other masonry exterior facades with window openings of relatively small size. Even the pioneering structural and architectural innovations by the Chicago School of Architecture did not basically change the relationship between the structural skeleton and the exterior facade. Although some of the buildings built in the early 1900's, inspired by the Chicago School, did have large window openings between the structural columns, the main structural skeleton was nevertheless completely enclosed and hidden behind masonry or stone cladding. It is therefore not surprising that for all practical purposes severe weather changes, such as those experienced in cities like Chicago and New York, did not cause any undue vertical movements resulting in structural or architectural damage in these buildings. Naturally, until very recently temperature effects on tall structures were seldom considered a design factor.

After World War II a new trend in architecture, reflecting the technology of steel and glass, suddenly made the temperature effects on tall buildings a significant factor in design. Metal and glass curtain walls replaced the traditional masonry or stone cladding. As the exterior expression of these buildings became lighter and lighter, it became more rational and architecturally more desirable to expose the main structural frame as a part of the exterior facade. However, it was soon apparent that this new expression of technology in architecture brought with it a new problem of temperature movement.

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TYPES OF COLUMN EXPOSURE

Depending on the structural and architectural resolution in a tall building, the exterior steel columns and spandrels are exposed primarily in three categories. These are:

1. Partial Exposure—In many buildings the structural columns project out only partially beyond the exterior glass line. Steel, aluminum, or a non-metallic cladding protects the projecting column from air and moisture from outside. This type of detail probably represents the majority of high rise steel buildings. For up to about 20 stories, the partial projection detail shown in Fig. 1 normally is not sufficiently affected by ambient temperature changes to require special treatment. Therefore, no further special care need be taken. Even for such details an approximate estimate of temperature movement of exterior columns should be made. (If the maximum movement at the top of the column relative to the interior column does not exceed ¼-in. under the worst weather conditions, no design or remedial corrections seem to be necessary.) Partially exposed buildings taller than 40 stories must be properly analyzed for vertical movements of the exterior columns, because in most recent cases such columns have indicated need for special consideration.

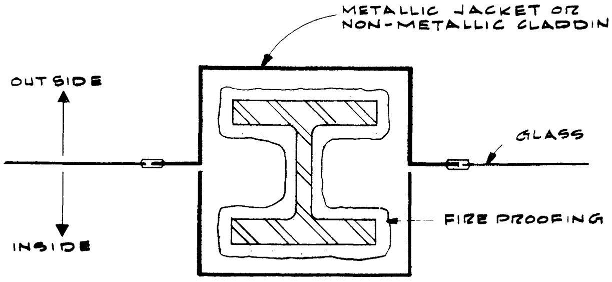


Figure 1

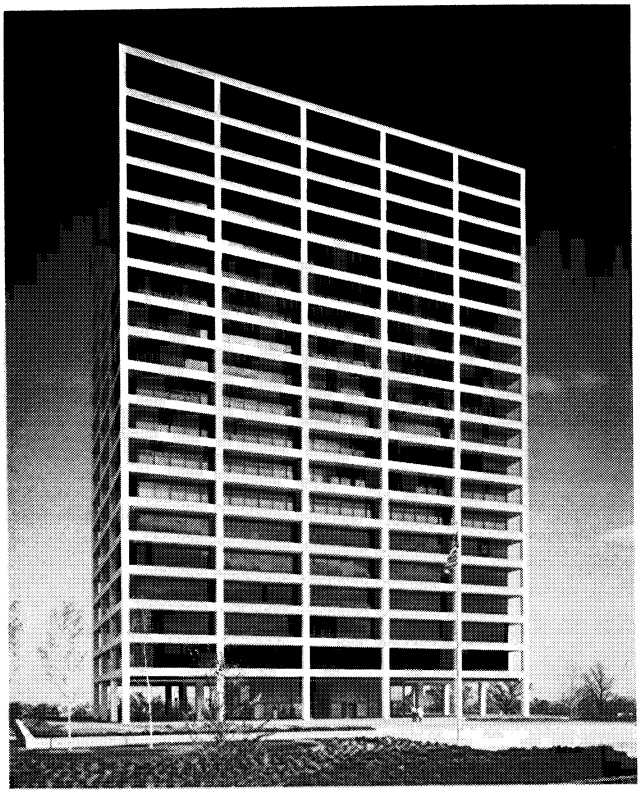


Figure 2

2. Fully Exposed Column—A large number of recent buildings have been built with a gallery type of curtain wall construction (for example, the BMA Building in Kansas City (Fig. 2), the First National Bank Building in Houston, and the U.S. Steel Building in Pittsburgh). It is obvious that if the exterior glass wall is sufficiently clear of the exterior columns, normal insulation and cladding will have practically no effect on the column temperature. In such cases the average column temperature would closely follow the average ambient temperature, although radiation from sunlight on the cladding may complicate the situation. It is understandable that this kind of exposed exterior column will require special structural and architectural details to compensate for vertical movement in buildings over only a few stories high. Almost invariably, all such buildings must be thoroughly analyzed and designed accordingly.

3. Exposure Due to Set Back Elevation—Although architecturally a set back elevation may not be a very desirable solution, many buildings are designed with set back elevations because of zoning requirements in certain cities. The effects of the set back elevation are different from (1) and (2) above only in that special care must be taken so that all aspects of the elevation of the building are analyzed separately.

TYPES OF DIFFERENTIAL MOVEMENTS

Partial or full exposure of exterior columns and spandrels result in differential movements in the building. Two types of differential movements must be considered:

1. Differential Movement Between Exterior Column and Interior Column—This is the most typical case to be considered. The differential movement is primarily caused by the temperature changes in the exterior columns through the winter and summer seasons. In the winter, while the interior column is maintained at a steady temperature of about 72°F the average temperature of the exterior column may drop to 30°F or even lower, depending on the type of exposure. This causes the exterior column to move down at each floor relative to the interior columns, the movement being cumulative and therefore maximum at the top floor. In the summer, of course, the reverse situation occurs. The exterior columns move upward relative to the interior columns, causing similar problems only in the reverse direction.

2. Differential Movements Between Adjacent Exterior Columns—Although this does not seem to be a typical case in every building, this situation arises at least at the four corner columns where, because of the cladding details, the nature of exposure of the typical exterior columns is substantially different. In a relatively short building this discrepancy would probably be minor in nature and can be overlooked. However, in a building over 40 stories high, the relative movement between the corner column and the adjacent exterior column may be sufficient to cause considerable structural load redistribution and, furthermore, cause possible failure of the curtain wall system in those areas. When the building plan is not rectangular, but of other irregular shape, every change in direction of the exterior cladding will similarly create a non-typical case of exposure (Fig. 3). To overlook this particular type of differential movement may create considerable problems requiring expensive repair and corrections after construction.

CLADDING CONSTRUCTION TYPES

As far as the temperature effect is concerned, the cladding material, whether steel, aluminum or other non-metallic material, does not substantially affect the movements of the column due to temperature exposure. However, the construction detail of such cladding may substantially affect these movements. The most common types of cladding detail may be classified in the following groups:

1. Cladding over Columns Encased in Concrete for Fireproofing—This type of cladding is used where the exterior steel columns are primarily fireproofed by encasing them in concrete, or where steel cladding is

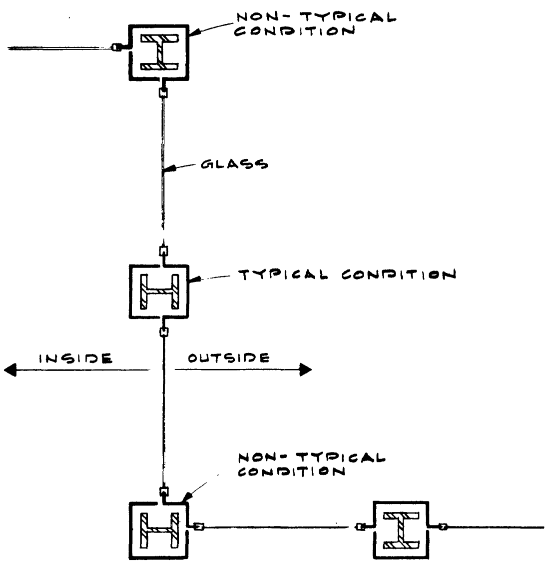


Figure 3

designed to act compositely with the encased concrete. When the cladding material is non-metallic or metallic acting compositely with the concrete encasement, as shown in Fig. 4, the temperature gradient through the columns cannot be established on the basis of the tests reported by McLaughlin in Part 1 of this paper. For an effective but simple temperature gradient analysis the method proposed by Fintel and Khan[2] can be used. However, the insulation value of the air gap between the cladding and the encasement concrete should be considered by simulating an effective increased dimension of the concrete encasement. In recent years a number of tall buildings have used the composite skin concept of steel cladding (Fig. 4). From an architectural point of view as well as from the point of view of water infiltration into the building, the seamless composite skin has been a great success. It should be pointed out that such composite skins also increase the structural stiffness of the exterior beams and columns. The temperature gradient analysis of concrete encased columns with composite steel skin is in some respects simpler than the non-composite skin in that no additional readjustment of the section has to be made. However, in view of the fact that often such a composite skin is painted in dark colors, an effective increase of surface temperature should be considered for making an analysis for summer conditions.

2. Insulated Cladding with No Provisions for Air Circulation—The response of the exterior column to the temperature exposure can be substantially reduced by attaching insulation either directly on the column or attaching insulation directly to the cladding material as shown in Fig. 5. The temperature gradient analysis for such a cladding detail depends on the correct evaluation of the insulating properties of the air space and the insulating material itself. In most cases a steady state analysis can be made by disregarding the existence of the steel column and simply determining the gradient through the air space occupied by the column itself. Because the conductivity of steel is extremely high in proportion to the conductivity of the air space, the gradient of the steel system will primarily follow the gradient of the air space surrounding it. Analysis of this kind of cladding cannot be accurately made without consideration of the three-dimensional effect of heat transfer, which will include the joint between the column and the spandrel at ceiling level. It is therefore recommended that, where accuracy is imperative, a full scale model test should be conducted to establish a realistic temperature gradient.

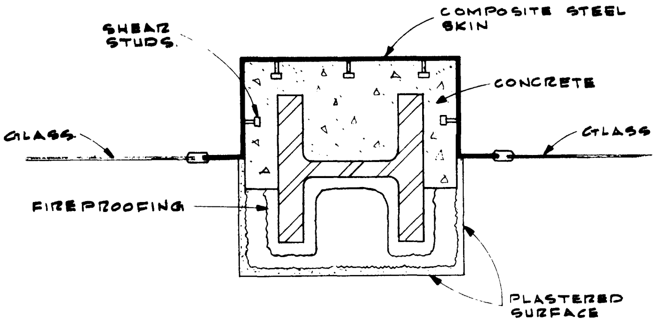


Figure 4

3. Cladding without Insulation, but with Air Space, and without Circulation—In terms of temperature effects on exterior columns, this type of cladding is undoubtedly the most ineffective and, therefore, unless the building is less than 10 stories high, should be avoided. In analyzing the temperature gradient of the steel section for this type of cladding construction, the points discussed in (2) above are valid in this case.

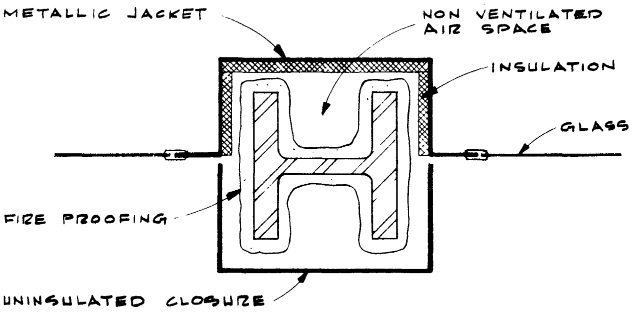


Figure 5

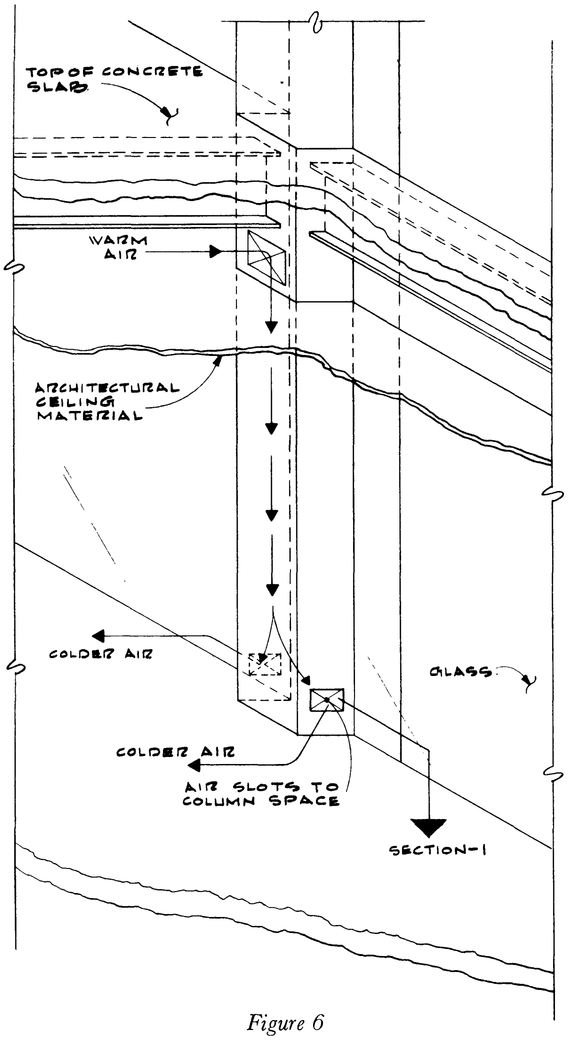


Figure 6

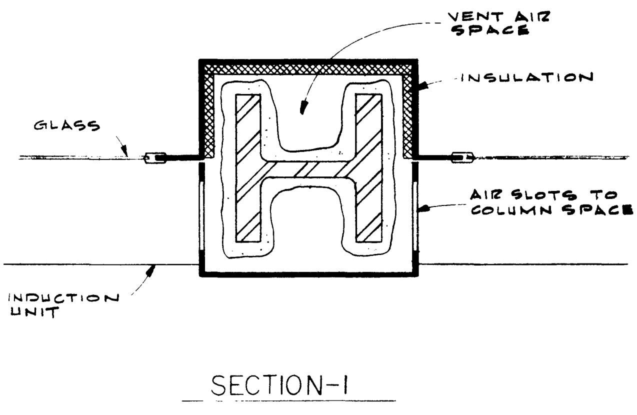


Figure 7

4. Insulated Cladding and Gravity Type Vertical Air Circulation through Column Space—Increasing the thickness of insulation on the inside face of the cladding beyond about 2 in. will not generally improve the temperature response of the column substantially, because the column temperature is still affected by the stagnant air temperature around the column. Therefore, if insulation of the exterior columns is a critical matter, as in a building over about 50 stories high, it is more effective to provide only an optimum thickness of insulation on the cladding material together with additional details to allow air circulation by gravity. This type of circulation was used recently in the 100-story John Hancock Center. In principle the detail consists of an air slot on the column jacketing inside the building at the induction units and also an air slot through the column jacketing in the ceiling space, as shown in Figs. 6 and 7. By providing these openings at the top and bottom of the column, cold air in the column space flows down into the induction units, whereas warm air replenishes the space from the ceiling area, thereby keeping the temperature of air around the column space very close to the inside temperature of the building. Full scale mock-up tests of this detail for the John Hancock Center, as shown in Figs. 6 and 7, indicated that a thickness of insulation of only 2 in., combined with the gravity air circulation, will maintain the temperature of the exterior columns within 5°F of the temperature of the interior columns under the worst winter exposure of $-10°F$. Since this type of detail confines the air circulation within the floor, this not only avoids fire spread and acoustic problems between floors, but also makes the system dependent on the inside temperature of the building.

5. Insulated Cladding with Forced Mechanical Ventilation to Control Temperature—Although there seems to be no technical reason why air could not be mechanically forced into the column to maintain steady temperature of the exterior columns, or electrical heating elements could not control the temperature of such columns, the authors doubt whether any system of temperature control should be dependent on an external mechanical or electrical system which may malfunction just when it is needed most. Furthermore, inasmuch as the building management is bound to change over the years, the importance of such mechanical control systems may not be duly impressed upon the new management personnel when a changeover occurs. It is therefore recommended that mechanical ventilation of the columns be avoided if there is some way of developing a gravity circulation system.

DESIGNING FOR TEMPERATURE EFFECTS

Prior to establishing methods of compensating for temperature effects, a preliminary assessment of the magnitude of expected responses should be made. Optimum design will depend upon a proper determination of exterior (ambient) temperature, average steel temperature, steel temperature change, temperature gradient, tolerable movement, free movement, restrained movement, method of analysis, and load factors.

Exterior Temperature—For steel members of usual size range, it is considered reasonable to use the minimum mean hourly temperature with a frequency of recurrence of once in 50 years as the equivalent steady state exterior winter or summer temperature for design purposes. A statistical analysis of weather bureau records for the locality of the structure will best provide this data. (See Fig. 8 for New York City.)

Steel Temperature (T_s) and Steel Temperature Change (Δ_t)—Methods recommended earlier can be used for determining average temperature, T_s, of the steel section for the required minimum exterior temperature, T_0, and the inside temperature, T_i, of 78°F. Steel temperature change $\Delta_t = T_i - T_s$.

Sample Calculation:

Column exposure condition: Face exposed, no fireproofing, no insulation.

Using curves developed in Part 1,

$$E = 0, \quad T_i = 78°\text{F}, \quad T_0 = -1°\text{F}.$$

From Eq. (5), Fig. 6 (Part 1):

$$T_s = T_0 + [0.41 - 0.24(E - 0.08)^2](T_i - T_0)$$
$$= 33°\text{F} \pm$$

$$\Delta_t = 78 - 33 = 45°\text{F}$$

Temperature Gradient—The outside flange of the column is subject to a constant lower temperature (t_2) than the inside flange temperature (t_1). Since the faces are subjected to different temperatures, a curvilinear gradient is established. The gradient produces a bowing moment in the column and must be considered. (See Fig. 13, Part 2.) Equation (1) and Fig. 7 in Part 1 can be used to determine the flange temperature differential for a bare beam:

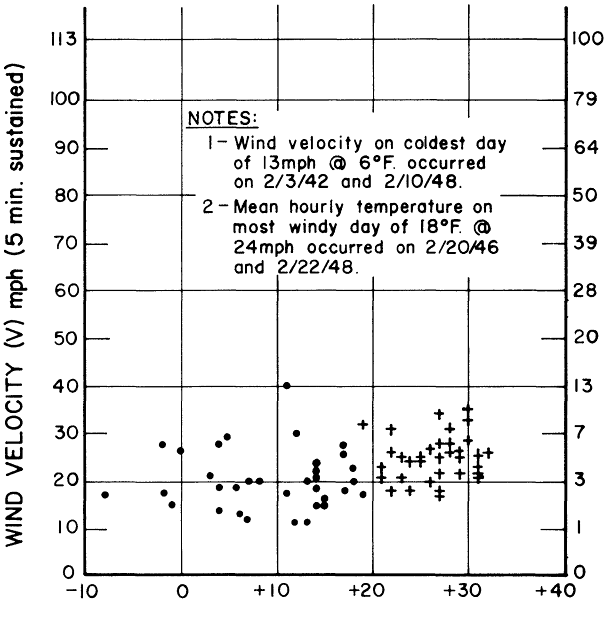
$$\Delta_t = [0.55 - 0.30\,(E - 0.25)^2]\,(T_i - T_0)$$

Tolerable Movement—The thermal movement of the exposed columns causes vertical relative displacements between exterior columns and interior columns. (See Figs. 5 and 9, Part 2.) These relative displacements are greatest in the upper stories and cause racking of floor slabs and distortion of partitions. Obviously, if large relative displacements must be reduced by restraining the movements, temperature-induced stresses in columns, beams, and connections must be accounted for.

Although it is possible to accommodate structurally any degree of restraint to thermal movements, cost consideration will generally require that the structure provide the least restraint possible.

It is recommended that the thermal movement of exterior columns relative to interior columns be limited to ¾-in. or $L/300$, whichever is smaller:

$$|\sigma_e| - |\sigma_i| \leq L/300 \text{ or } \tfrac{3}{4}\text{-in.}$$



NOTES:

1 - Wind velocity on coldest day of 13mph @ 6°F. occurred on 2/3/42 and 2/10/48.

2 - Mean hourly temperature on most windy day of 18°F. @ 24mph occurred on 2/20/46 and 2/22/48.

• WIND VELOCITY ON THE COLDEST DAY OF EACH FEB.

\+ MEAN HOURLY TEMP. ON THE MOST WINDY DAY OF EACH FEB.

HIGHEST RECORDED WIND IN NEW YORK CITY IS 113MPH.

Figure 8

where

$|\sigma_e|$ = exterior column displacement

$|\sigma_i|$ = interior column displacement

L = distance between columns

Free Movement—Free movement of exposed columns in steel framed buildings 20 to 30 stories in height may produce relative displacements which can be tolerated by the partitions and building finishes, even if steel temperature change is severe. For a 10°F steel temperature change, Δ_t, free displacement of a steel exterior column in a 300 ft high building is approximately ¼-in.:

$$\Delta_l = \Delta_t \alpha l$$
$$= 10 \times \frac{0.00065}{100} \times 300 \times 12 = 0.234 \text{ (say } \tfrac{1}{4}\text{-in.)}$$

where

α = coefficient of expansion of steel

l = height of column, in.

Assuming no restraint from floor framing (simple connections), an exterior steel column temperature change of 30°F would have to develop before restraint of movement need even be considered. Furthermore, unless the exterior columns must function as part of the wind bracing requiring rigid connections to the exterior columns, temperature induced stresses cannot develop.

Only bowing stresses due to temperature gradient need be investigated.[1] If free movement results in relative displacements which cannot be tolerated (greater than ¾-in. to 1 in.), then the exterior columns must be restrained and temperature induced stresses must be investigated.

Restrained Movement—In multistory frames with exposed exterior columns, the effect of restrained temperature movement of the exterior column is significant only in the elements of the exterior bay. These elements are the exterior column, the exterior bay beam, and the first interior column. After the decision to restrain movement is reached, procedures for guiding the designer in selecting suitable column and beam stiffness are necessary.

The following is a suggested procedure to guide the designer to quickly determine preliminary member sizes:

1. Determine minimum mean hourly exterior (ambient) temperature for locality of structure.
2. Determine column exposure condition to fit curves developed in Part 1 or as recommended in this paper.
3. Select average steel temperature T_s from the appropriate curve for the required minimum exterior temperature T_0.
4. Determine steel temperature change $\Delta_t = T_i - T_s$.
5. Determine temperature for a bare column from Fig. 4, Part 1.
6. Calculate free movement and establish the percent of free movement to be permitted.

   Example:

   60 story building, 800 ft high.

   $\Delta_t = 40°$ F

   $\Delta_l = 40 \times 0.00065/100 \times 800 \times 12$
   $= 2.5$ in. (free movement)

   Tolerable movement of 0.75 in. = 0.75/2.5 or 30% of total free movement.
7. Determine relative axial column stiffness and beam stiffness to restrict residual movement between exterior and interior column to 30 percent of free movement. (Use curves for predicting residual relative movement.)[1]

   For quick preliminary design, the original curves in Ref. 1 can be used. These curves are reproduced in Figs. 9 and 10 for the reader's convenience.

   Example:

   60 story building, 800 ft high.

   $\Delta_t = 40°$F

   Residual movement = 30% of free movement.

   a. Determine preliminary column sizes for dead and live load in the exterior bay. (Include wind load if part of wind system.)

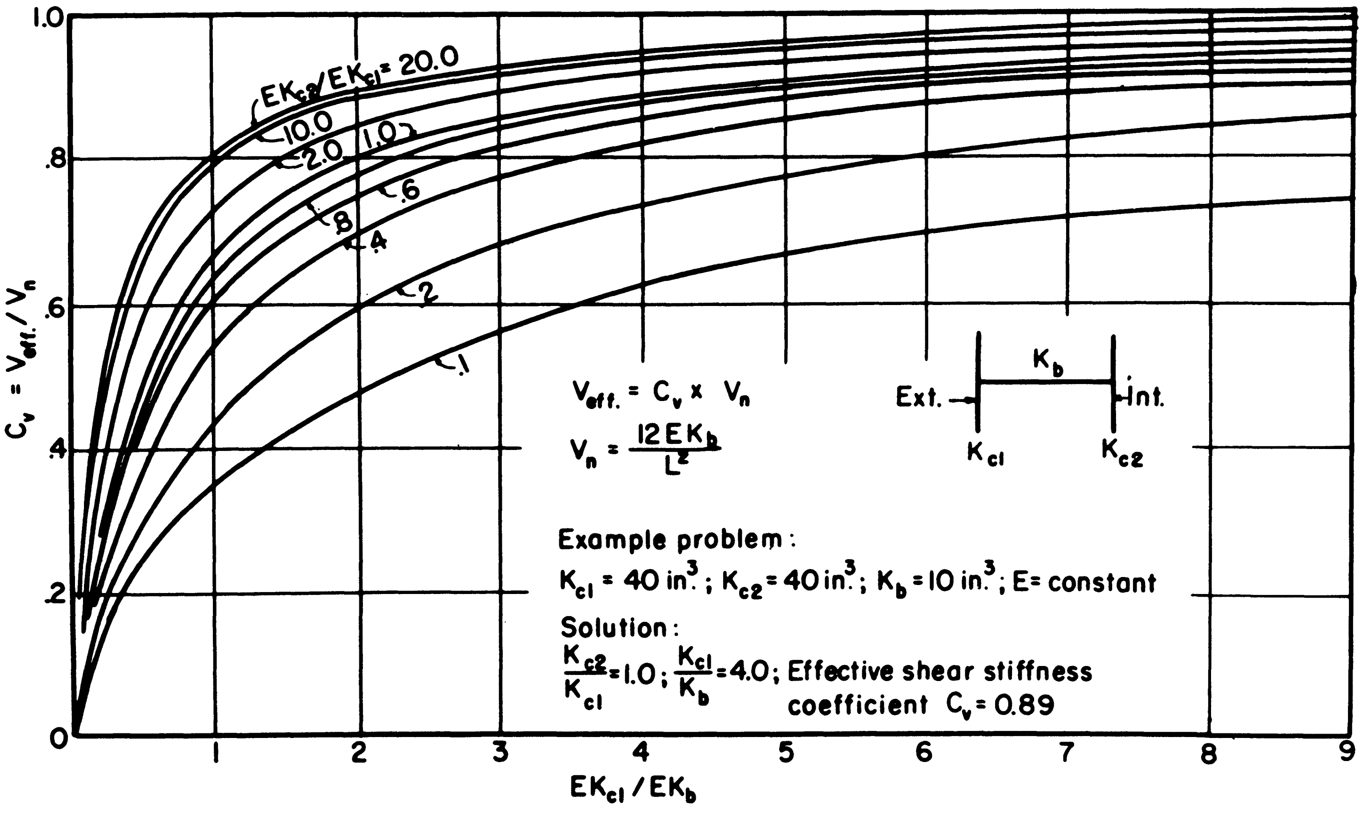


Figure 9

b. Using Fig. 10, enter appropriate chart:

If $V_{c(Bot)}/V_{c(Top)} = 10$ and
V_{eff} = constant @ 0.2 from ground,
$(V_{eff}/V_c)_{10} = 0.60\pm$

For 60 story building, $V_{eff} = 0.60/(60/10)^2 \times V_c$
$= 0.017\ V_c$ @ 0.6 from ground
$(V_{eff}/V_c)_{10} = 0.40\pm$

For 60 story building, $V_{eff} = 0.40/(60/10)^2 \times V_c$
$= .011 V_c$ @ 1.0 from ground,
$(V_{eff}/V_c)_{10} = 0.30\pm$

For 60 story building $V_{eff} = 0.30/(60/10)^2 \times V_c$
$= 0.008\ V_c$

c. Using values from (b) above, approximate values for beams can be established at each floor level.

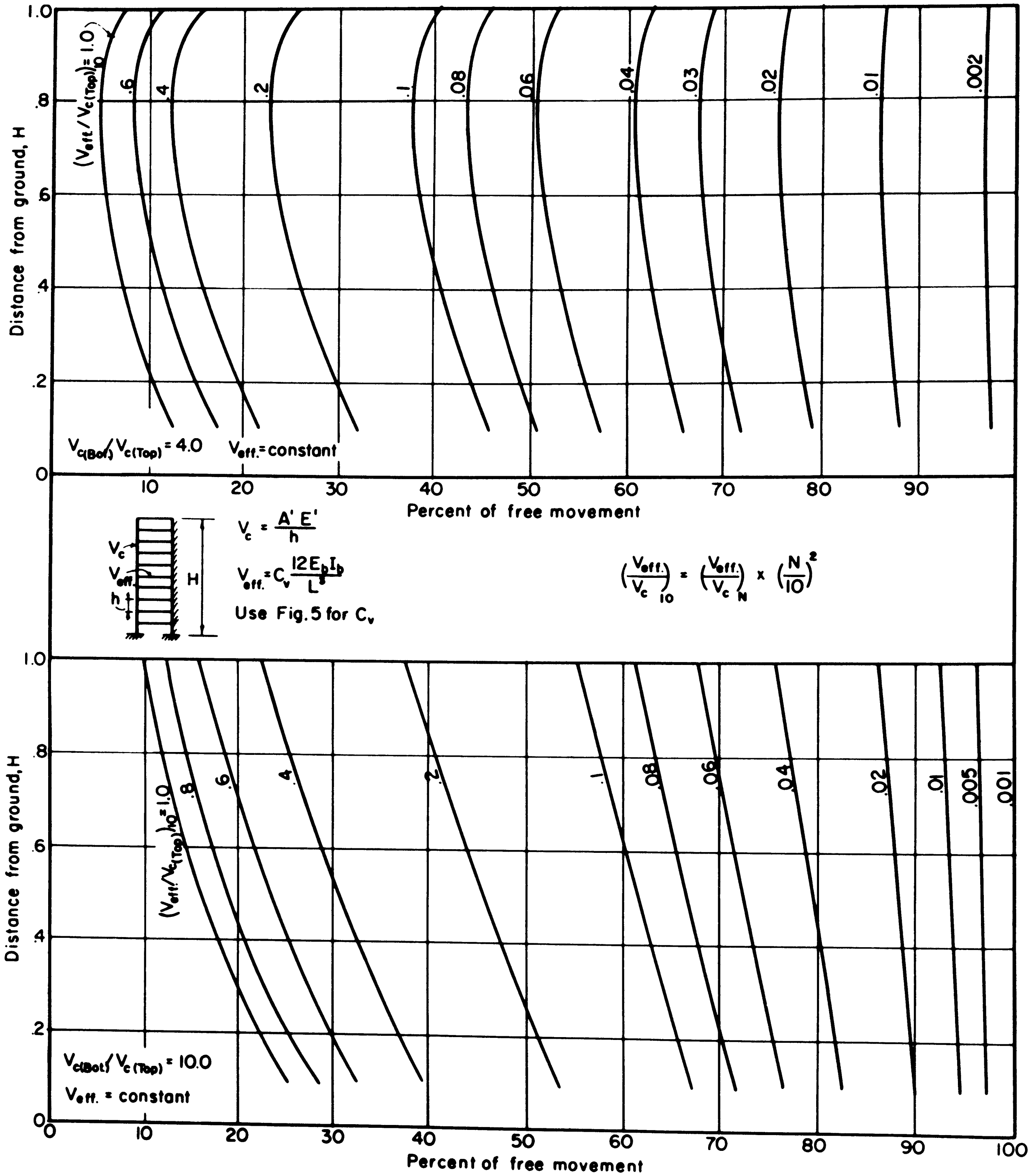


Figure 10

Method of Analysis—After preliminary design by the Khan and Fintel method[1], the final design can be based on computer programs for determining displacements and member forces due to temperature induced loading. Single bay model analysis, double bay model analysis, and complete Strudl analysis are completely discussed in Part 2.

Iterative methods of analysis by manual calculation are also available and can be used to obtain fairly accurate solutions within two or three cycles for buildings having relatively flexible girders.[1]

Load Factors—Steel stresses due to restraint to temperature induced length changes, load transfer, and bowing should be combined with effects of gravity loads and lateral loads such as wind or earthquake. It would be unrealistic to design a structure for the full combined effect of lowest temperature, highest wind velocity, and gravity loads. Furthermore, although extreme low temperature conditions may exist for a longer period, coincidence of full temperature and highest wind effects is extremely improbable during the lifetime of a structure (50 to 100 years). On the basis of these considerations, it is proposed that the working stress design be based upon the following formulas:

$$D + L + T + C_1 W = 1.33S$$
$$\text{or } D + L + C_2 T + W = 1.33S$$
$$\text{or } D + L = 1.0S$$

In the above formulas D, L, T, and W are the effects of dead load, live load, temperature, and wind, respectively, S is the allowable working stress, factors C_1 and C_2 depend on local climatic conditions.

To assess the coincidence of high wind and extreme temperature and factors C_1 and C_2, the wind velocity of the coldest day in February in New York City available for 35 years has been plotted in Fig. 8. Also, mean hourly temperatures available for the most windy days in February for 35 years have been plotted on the same chart. The ordinate on the right side shows percentages of wind load for various wind velocities, based on the assumption that the highest wind of 113 miles per hour ever registered in New York City represents 100 percent of the wind load. It is also assumed that the relation between wind load and wind velocity is:

$$P = 0.00256V^2 \times SF \times (Fg)^2$$

where

P = wind load in psf
SF = shape factor
V = wind velocity in mph
Fg = gust factor

It can be seen that the strongest wind (40 mph) associated with lowest February temperatures corresponds to only 13 percent of the maximum wind load.

This example study of wind and temperature for New York City leads to the conclusion that factor $C_1 \fallingdotseq 0.13$ and factor $C_2 = 0.10$ may be used in the above equations for that city.

If it is necessary to determine C_1 and C_2 more accurately, the wind and temperature data must be converted to wind and temperature stresses for each member and the maximum total stress levels must be determined for each member to be designed.

Methods of Compensating Temperature Effects—The obvious method of compensating temperature effects would be to move the curtain wall outside the exterior column system, so that in effect the exterior column is completely within the building environment. This, of course, is possible only when the building is designed to have a full traditional curtain wall skin, in which case only a few inches of space between the fireproofed exterior column and the curtain wall would adequately maintain the exterior column within the environment of the building. However, this is neither architecturally acceptable in some cases, nor is it economical for certain types of structural systems where the exterior columns may provide direct support for the window wall system. Where exterior columns are partially exposed, one or more of the following methods of compensating temperature effects may be adopted:

1. *Insulated Cladding with Gravity or Mechanical Ventilation*—As discussed earlier, this is probably the simplest method of compensating for temperature effects. However, if the column projection outside the curtain wall is unusually large, this method of correcting temperature movement may not be feasible from the point of view of heat loss and condensation.

2. *Rigid Beam to Column Connections*—In a rigid frame type construction, beams spanning between exterior and interior columns and rigidly bolted or welded to these columns will provide resistance to relative movement of the exterior column. By adjusting the stiffness of these connecting beams, the free movement of the exterior columns can be substantially controlled. In such cases, however, the additional stresses in the connecting beams, as well as in the columns, must be incorporated in the design of the structural members as discussed previously. In addition, redistribution of loads between the exterior and interior columns for both winter and summer conditions must be considered in the design of the columns.

3. *Restraining Truss at Top of Building*—The vertical movement of the exterior column being cumulative and maximum at the top of the building, the relative movement of the exterior and interior columns can be effectively reduced by providing a restraining truss connecting the exterior and interior columns across the

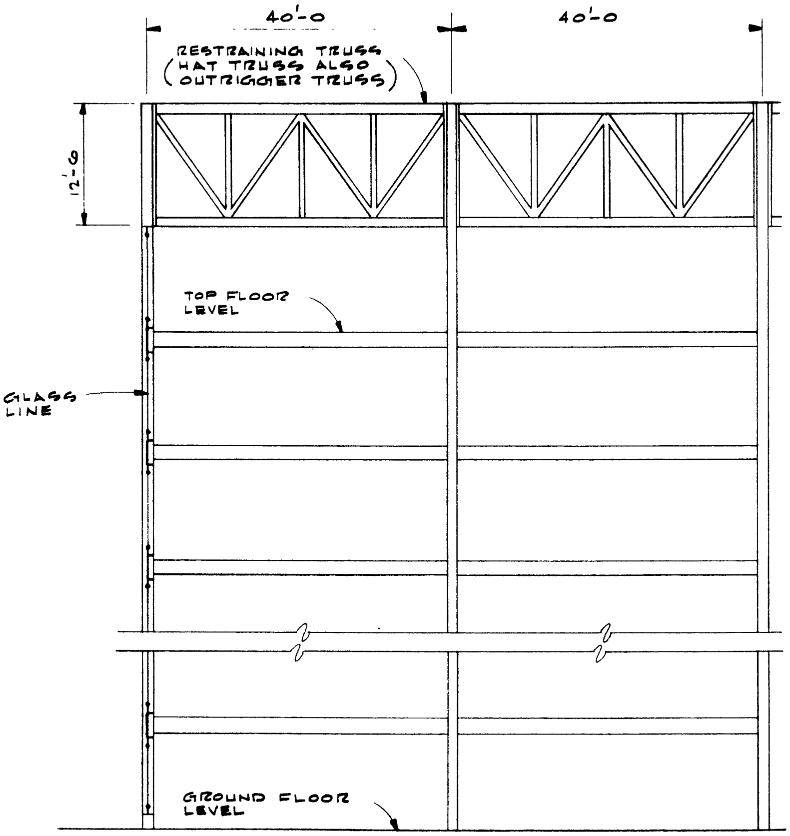


Figure 11

building section. Typically, this truss is often a part of the primary structural system for a tall building, as shown in Fig. 11, the main purpose of which is to reduce lateral sway under wind load. The restraining trusses are generally one story deep and therefore very rigid. Such trusses can therefore practically eliminate the relative movement between the exterior and interior columns.

However, it should be remembered that the design of such trusses, even if primarily for increasing the stiffness of the building against wind load, must include the load transfer from the exterior to the interior column. For structural reasons, restraining trusses are sometimes provided both at the top and at mid-height of the building as was used for the BHP Building in Melbourne (Fig. 12). In such cases the restraining trusses should be designed for redistribution of the forces between the exterior and interior columns due to relative temperature difference between these columns from the top to the mid-height restraining trusses.

4. *Partition Design Criteria*—Even though partitions are not structural elements, the relative movements of the exterior columns with respect to the interior columns may be sufficient to damage the normally used partition systems. The story distortion as shown in Fig. 13 induces shear and bending stresses in the partitions and, if not relieved by proper details, can cause unsightly as well as acoustically unacceptable cracking. In order to avoid such cracking of partitions, details around the edges of partitions should be developed to allow vertical as well as horizontal slippage. One of the simplest ways to achieve this is to provide a channel enclosure for the partition walls at the columns and the ceiling. The detail, shown schematically in Fig. 14, has been effectively used in the design of the John Hancock Center.

Structural Details—Two cases of special structural details need to be further discussed. In one case, relative movement is controlled by restraining truss; in the other, the relative movement is uncontrolled by deliberately releasing all fixity of the beams at the columns.

1. *Restraining Trusses*—The restraining trusses should be designed for complete reversal of stresses to take care of load redistribution both in summer and winter. Therefore, all the diagonal and chord members should be designed for maximum compression forces caused by a combination of wind and temperature effects. The connection of the top and bottom part of the truss to the exterior column should be designed to resist horizontal forces to compensate for the tendency of the exterior column to bow out due to temperature gradient.

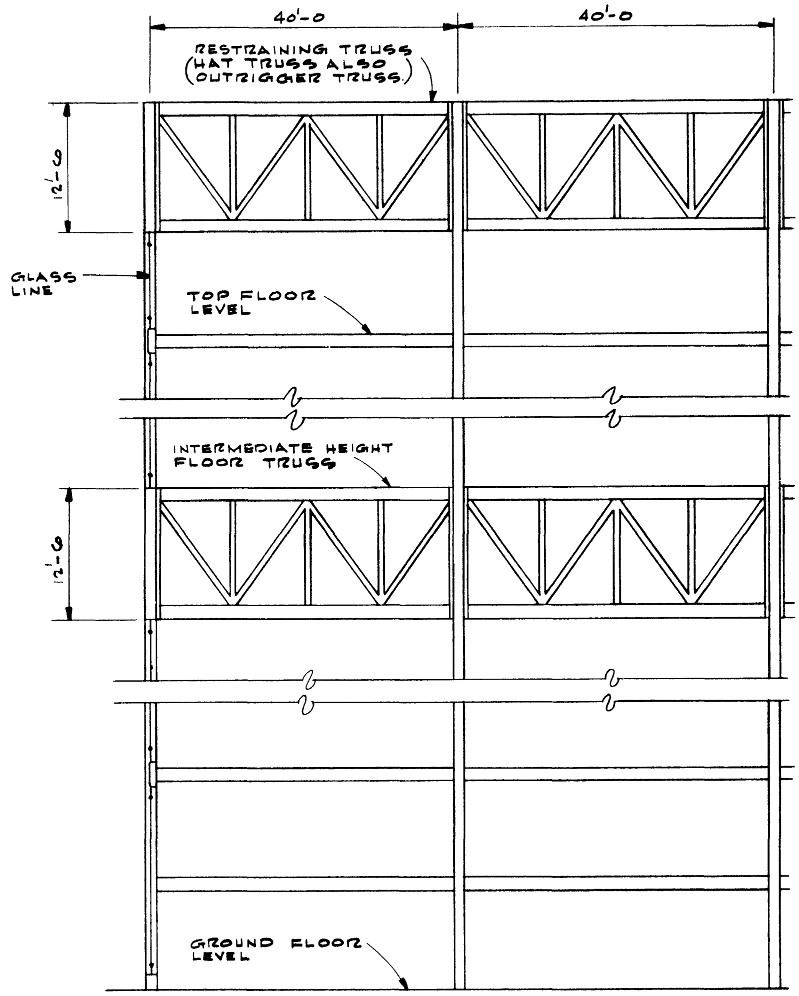


Figure 12

2. *Simply Supported Beam Connections*—If the expected relative movement between exterior and interior columns is more than ¾-in., special care should be taken to design the beam-to-column connection to provide free rotation of the joint without causing appreciable moments. Ideally, the beam-to-column shear connectors should be designed for unrestrained rotation. Welded type shear connectors should therefore be avoided. If bolted type shear connections are used, the bolt holes should be horizontally slotted and high strength friction bolts should be avoided. However, because the beams connected to the columns must also provide lateral stability to the exterior columns, it is advisable that these connections be designed to slip and rotate only under higher forces, but develop the necessary restraining axial force for the stability of the columns under normal loads. A typical schematic detail is shown in Fig. 15. It is apparent that the total connection should consider the overall restraining capacities of the main shear connection as well as the actual restraining capacity of the floor slab, which generally extends to the glass line.

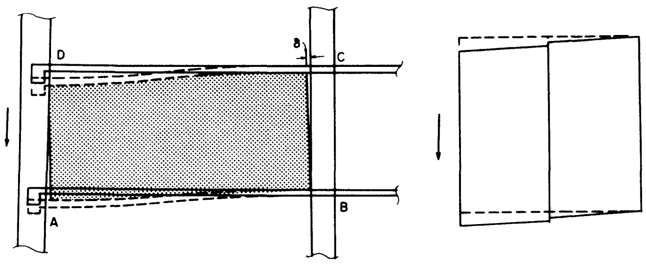
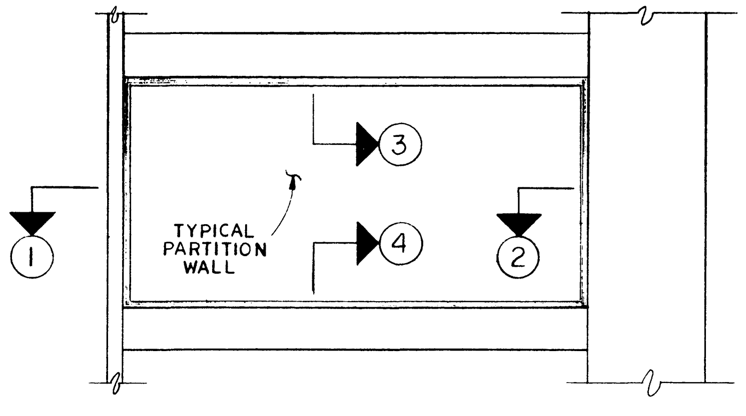
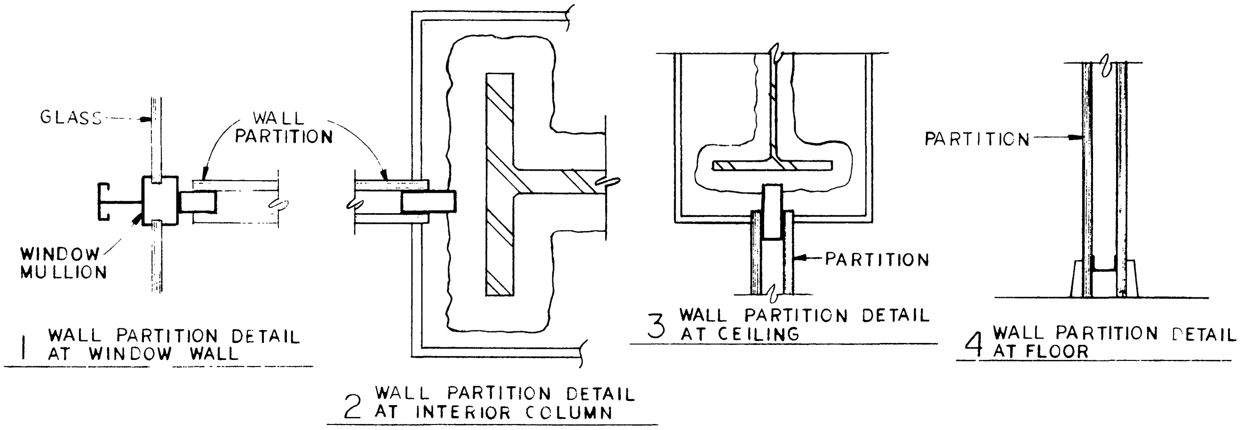


Figure 13



PARTITION ELEVATION



1 WALL PARTITION DETAIL AT WINDOW WALL

2 WALL PARTITION DETAIL AT INTERIOR COLUMN

3 WALL PARTITION DETAIL AT CEILING

4 WALL PARTITION DETAIL AT FLOOR

Figure 14

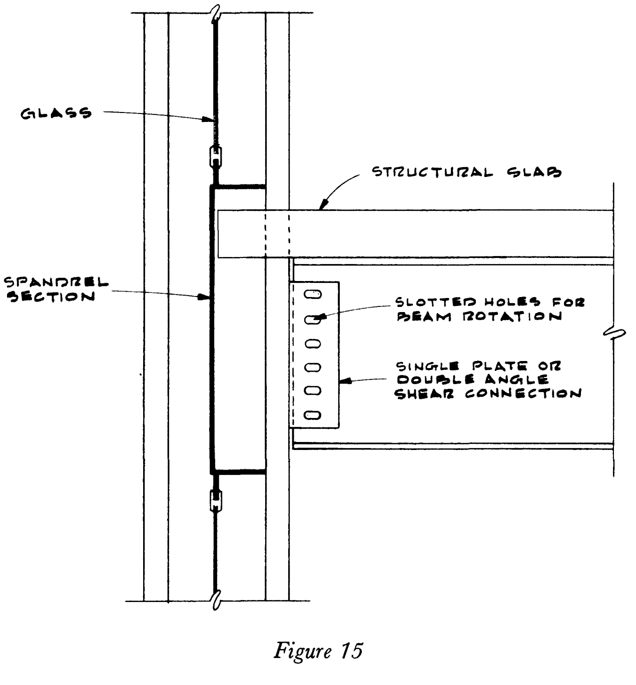


Figure 15

CONCLUSIONS

A number of significant design considerations for temperature effects on tall buildings have been discussed. Effort has been made to correlate the results of Parts 1 and 2 of the paper into the development of practical design principles and structural details. Actual building examples have been given to bring forth some of the principles in this paper. It is expected that, together with Parts 1 and 2, this paper will give the design engineer an insight into the problems of temperature effects on tall buildings and provide him with sufficient guidance to develop practical designs related to the total architectural development of a tall steel framed building.

REFERENCES

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