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TEMPERATURE EFFECTS IN MULTI-STORY BUILDINGS

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This paper presents an analytical study to investigate the effect of daily and seasonal temperature changes on the multi-story concrete structures. Two and three-dimensional calculations have been carried out by applying a temperature loadings on concrete structures, taken into consideration different cases of structure configurations and support conditions. The temperature loading were either a uniform drop of temperature or linear gradient temperature. The results were investigated and compared to each other.

KEYWORDS: *Daily temperature; Seasonal temperature; Multi-story; Concrete buildings*

NOTATION

A	Area of cross section, m^2	R	Degree of restrain
b	Width of cross section, m	T	Ambient temperature
E_c	Modulus of elasticity of concrete, kg/cm^2	***Greek Symbols***	
h	Depth of the cross section, m	α	Coefficient of heat expansion, 1/°C
H	Floor height, m	ε_a	Axial strain at centroid
I	Second moment of inertia, m^4	ε_T	Free thermal strain
M	Bending moment, Ton.m	ε_T	Restrained component of thermal strain
N	Axial force, Ton	ϕ	Curvature
n	Depth of natural axis, m	σ_n	Net stress
P	Restraining axial force, Ton	ΔT	Change in temperature, °C

1- INTRODUCTION

Concrete Structure are subjected, besides live and dead loads, to seasonal and daily temperature changes and consequently temperature loads as a result of there interaction with the surrounding environment and due to their exposure to solar radiation. Such temperature loadings consequently lead to thermal stresses in most structural elements. These thermal stresses can be comparable or even exceed in magnitude the stresses induced by the live and dead loading in case of no or little thermal insulation and could lead to severe damages if not considered during the design phase.

All concrete elements and structures are subject to volume change in varying degrees, dependent upon the makeup, configuration, and environment of the concrete. Uniform volume change will not produce cracking if the element or structure is relatively free to change volume in all directions. Concrete contraction or expansion is the algebraic sum of these volume changes.

In reinforced concrete elements, the primary concern is with these volume changes resulting from thermal and moisture changes [1]. Other volume changes are alkali-aggregate expansion, autogenous shrinkage, and changes due to expansive cement. Autogenous shrinkage is the volume change due to the chemical process that occurs during hydration.

The change in temperature to be considered in the design of reinforced concrete elements is the difference between the peak temperature of the concrete attained during early hydration (normally within the first week following placement) and the minimum temperature to which the element will be subjected under service conditions. The initial hydration temperature rise produces little, if any, stress in the concrete [1]. At this early age and during the temperature rise period, the concrete has a relatively low elastic modulus and the compressive stresses due to restrained expansion are easily relieved by creep. During cooling, the concrete matures and, when the thermal contraction is restrained, the tensile stresses generated are less easily relieved. These can be of sufficient magnitude to cause cracking which commonly occurs at the half or one-third points along a bay. By assuming a condition of no initial stress, a slightly conservative and realistic analysis results. In the extreme case of a fully restrained element, a change in temperature of the order of only 10 °C can result in cracking [2]. Therefore, the high temperature rises which can result in long-term strength reductions are not essential to the promotion of cracks. However, if there was no restraint, the concrete would contract without cracking.

To restrain an action is to check, suppress, curb, limit, or restrict its occurrence to some degree. The degree of restrain, R, is the ratio of actual stress resulting from volume change to the stress which would result if completely restrained. Numerically, the strain is equal to the product of the degree of restraint existing at the point in question and the change in unit length which would occur if the concrete were not restrained.

All concrete elements are restrained to some degree by volume because there is always some restraint provided either by the supporting elements or by different parts of the element itself.

Restrained volume change can induce tensile, compressive, or flexural stresses in the elements, depending on the type of restraint and whether the change in volume is an increase or decrease. We are normally not concerned with restraint conditions that induce compressive stresses in concrete because of the ability of concrete to withstand compression. We are primarily concerned with restraint conditions which induce tensile stresses in concrete which can lead to cracking. The types of restraint are external restraint (continuous and discontinuous) and internal restraint. Both types are interrelated and usually exist to some degree in all concrete elements. According to BS code [2], values of external restrained factor R can be obtained from Table (1).

At early age of concrete, temperature changes affect concrete water content, environment relative humidity and consequently concrete creep and shrinkage. If creep and shrinkage response to temperature changes are ignored and if complete histories for concrete water content, temperature and loading are not considered, the actual response to temperature changes may drastically differ from the predicted one[3]. Most of researches are concentrated in studying the temperature effect in correlation with shrinkage and creep on concrete elements [5-10]. Few researches are extended to study the thermal response of a complete structure due to seasonal temperature [11-13].

Table (1): Values of external restraint recorded in various structures

Pour configuration	Restraint factor (R)
Thin wall cast on to massive concrete base	0.6 to 0.8 at base 0.1 to 0.2 at top
Massive pour cast into blinding	0.1 to 0.2
Massive pour cast on to existing mass concrete	0.3 to 0.4 at base 0.1 to 0.2 at top
Suspended slabs	0.2 to 0.4
Infill bays, i.e. rigid restraint	0.8 to 1.0

Temperature changes can significantly affect deflections of reinforced concrete building structures [4]. Deflections occur in unrestrained flexural members when a temperature gradient is set up between opposite faces of the member. In cases where deformations due to temperature change are restrained, tensile stresses induced in the member can result in cracking and consequent reduction in flexural stiffness. Because temperature effects do not often affect the ultimate limit state of the structure, effects of temperature on deflection are sometimes not considered in design.

Priestly (14) points out that an iterative solution is necessary to solve for the final strain distribution in cracked sections since the extent of cracking is a function of the thermal load. However for design, he suggests that unrestrained thermal curvature be calculated on the basis of the uncracked section, and that thermal continuity moments and secondary thermal stresses be calculated on the basis of moment of inertia associated with the expected distribution of cracking at service loads.

Mentes et al. (15) presented a method for estimating reduced stiffness due to thermal effects. The method is based on Branson's effective moment of inertia and an iterative procedure. Branson (16) has presented a method for calculating differential temperature effects in composite construction.

Designing for thermal and shrinkage stresses is the most neglected part of today's design practice. With the use of higher strength materials and more refined methods of analysis the need to consider temperature effects is becoming increasingly important.

The objective of this paper is to indicate the thermal influence of multi-story reinforced concrete buildings that result from ambient temperature change. The scope of the study is restricted to performance of structures in service. Temperature effects due to heat of hydration are not considered.

2- THERMAL BEHAVIOUR OF CONCRETE SECTIONS

For uniform temperature change, the free thermal strain can be calculated as follows:

$$\varepsilon_T = \alpha.\Delta T \qquad (1)$$

Where α is the coffeceint of heat expansion and ΔT is the change in temperature.

The restrained component of the thermal strain ε_R which will be accommodated by cracks is given by the following equation

$$\varepsilon_R = R.\alpha.\Delta T \qquad (2)$$

The induced thermal stress due to restriction is:

$$\sigma_T = R.\alpha.\Delta T.E_C \qquad (3)$$

Where EC is the modulus of elasticity of concrete.

For linear distribution of temperature in unrestrained cross section [3], Consider a temperature distribution T(y) on the cross section shown in Fig. (1). Thermal strain at distance y from the bottom of the section is given by:

$$\varepsilon_T(y) = \alpha.\Delta T(y) \qquad (4)$$

To restrain movement due to temperature T(y), apply a stress in the opposite direction to εT(y):

$$\sigma_T(y) = E_C.\alpha.\Delta T(y) \qquad (5)$$

The net restraining axial force and moment are obtained by integrating over the depth:

$$P = \int_A \sigma(y).dA = \int_0^h E_C.\alpha.\Delta T(y).b(y).dy \qquad (6)$$

$$M = \int_A \sigma(y).(y-n).dA = \int_0^h E_C.\alpha.\Delta T(y).b(y).(y-n).dy \qquad (7)$$

To obtain total strains on the unrestrained cross section, as shown in Fig. 2, apply P and M in the opposite direction to the restraining force and moment. Assuming plane sections remain plane, axial strain (εa) and curvature (φ) are given by:

$$\varepsilon_a = \frac{P}{A.E_C} = \frac{\alpha}{A}\int_0^h \Delta T(y).b(y).dy \qquad (8)$$

$$\phi = \frac{M}{E_C.I} = \frac{\alpha}{A}\int_0^h \Delta T(y).b(y).(y-n).dy \qquad (9)$$

The net stress distribution on the cross section is given by:

$$\sigma_n(y) = \frac{P}{A} \pm \frac{M.(y-n)}{I} - E_C.\alpha.\Delta T(y) \qquad (10)$$

For a temperature gradient varying linearly from 0 to ΔT, the curvature obtained by Eqn. (9) is given by:

$$\phi = \frac{\alpha.\Delta T}{h} \qquad (11)$$

The final thermal moments in the members are proportional to the degree of restraint R.

3. STRUCTURAL ANALYSIS

3. Two-Dimension Analysis on Multi-Storey Concrete Frames.

To study the influence of seasonal and daily temperature changes on multi-story concrete frames, the following cases in Table (2) were taken into consideration:

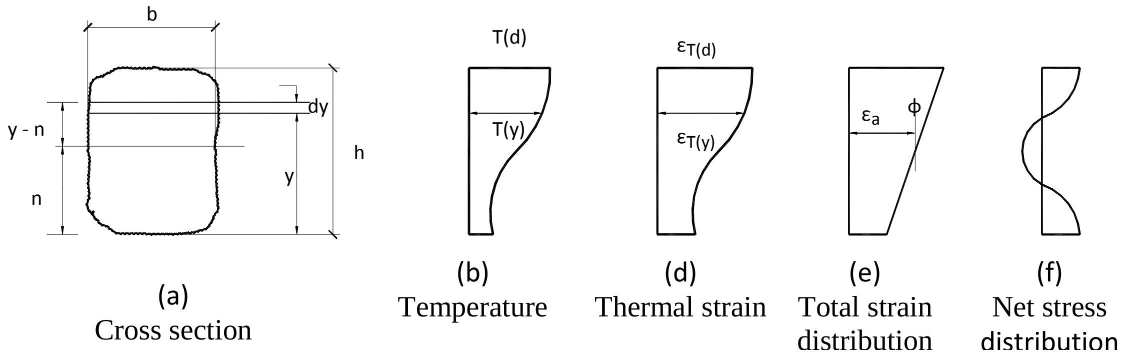


Fig. (1): Thermal stress and strain distributions on general cross section due to linear temperature gradient

Table (2): Cases of study

Case of study	No. of bays	Bay length (m)	No. of floors	Temperature distribution	Concrete strength (kg/cm^2)	Support condition
1	One bay	6.0	9	Uniform	250	Fixed
2	Three bays	6.0	9	Uniform	250	Fixed
3	Nine bays	6.0	9	Uniform	250	Fixed
4	One bay	6.0	5	Uniform	250	Fixed
5	One bay	10.0	9	Uniform	250	Fixed
6	One bay	6.0	9	Uniform	250	Hinged
7	One bay	6.0	9	Uniform	250	Hinged-Roller
8	One bay	6.0	9	Linear	250	Fixed
9	One bay	6.0	9	Uniform	400	Fixed

In all cases, height of floor is 4.0 m, dimension of column is 0.3x0.7 m, dimension of beam is 0.3x0.6 m. modulus of elasticity of concrete, Ec , is 2.35x105 kg/cm^2, except for case 9, Ec is 2.9725x105 kg/cm^2.

All frames are subjected to a uniform fall in temperature of 20 °C except for case 8 where only the upper beam was subjected to a linear temperature with 20 °C difference between outside, which has the higher value, and inside. These values were taken to simulate the actual conditions that the drop of the temperature produces tensile stress and consequently cracks. For linear temperature case, it simulates the effect of the sun heat in roof beams. The analysis was performed by SAP program. The results of different cases are shown in Fig. (2) to Fig. (11).

In general, in case of uniform fall of temperature, only columns and slab of the first floor which have the major influence. Above the third floor, the effect of temperature seems to be diminished. The first floor beam has a positive axial force and the second has negative one. When comparing the results of cases 1, 2 and 3 which are illustrated in figures from Fig. (2) to Fig. (6), it can be seen that as the number of bays increases, the axial and bending moment forces increase. For the first floor beams, the maximum axial forces were 1.52 ton, 5.6 ton (middle bay) and 21.6 ton (middle bay) for cases 1, 2 and 3 respectively. The maximum bending moments in the external columns were 2.61 ton.m, 7.62 ton.m and 18.16 ton.m for cases 1, 2 and 3 respectively. This means that by increasing the number of bays from one to three bays, the axial force and bending moment increased by 3.68 % and 2.92 %. Also, when the number of bays increased from one bay to nine bays, the axial force and bending moments increased by 14.21 % and 6.96 % respectively.

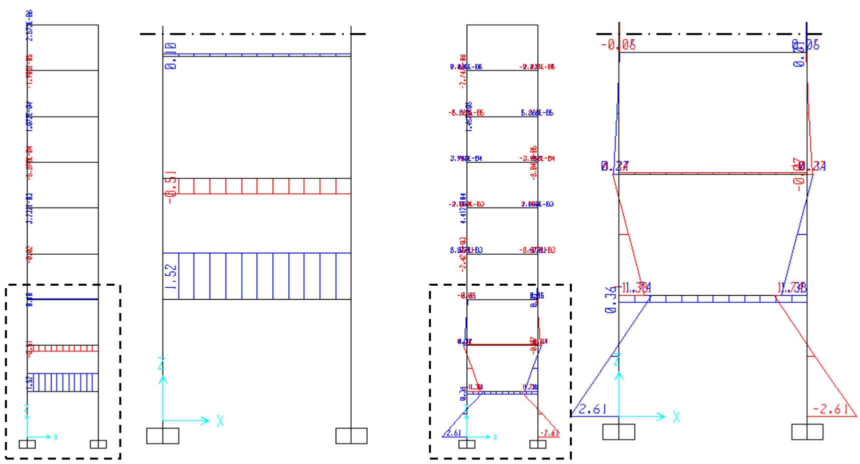
Figure (7) shows the results of five floors frame which represents case 4. It is clears from the figure that the resulted forces are the same as those for case 1. This means that the number of floors does not affect the resulted forces due to temperature drop.

The effect of bay length was investigated in case 6. Figure (8) illustrates the results for this case. When comparing the results of this case with the results in case 1, it is clear that as the bay length increases, the axial force and bending moment increase. The axial force and bending moment increased by 1.6 % and 1.61 % respectively when the bay length increase from 6.0 m to 10.0 m. These ratios approximately represent the bays lengths ratio which is 10/6 = 1.66.

If the base connection of the frame is changed, the results are drastically affected. This can be demonstrated by comparing the results of case 6 with case 1. When the base connection was changed from fixed as shown in Fig. (2) to hinged as shown in Fig (9), the axial force and bending moment were reduced to be 34 % and 38 % from those in case 1, respectively. For hinged-roller base connection, (case 7), zero axial force and bending moment were attained, even though the structure is statically indeterminate.

The effect of linear gradient temperature on the roof beam is shown in Fig. (10). The results indicate that only these floor which is close to the last floor is affected by the temperature changes. A constant bending moment over the beam length was encountered. As expected, the outside fibers of the beam were exposed to tensile stress. The axial force in the last upper beam is compressive one.

The effect of temperature fall on high strength concrete frame is illustrated in Fig. (11). By comparing the results with those in Fig. (2), it is clear that the axial forces and bending moment increases as the concrete compressive strength increase.



N F D B M D

Fig. (2): Normal force and bending moment

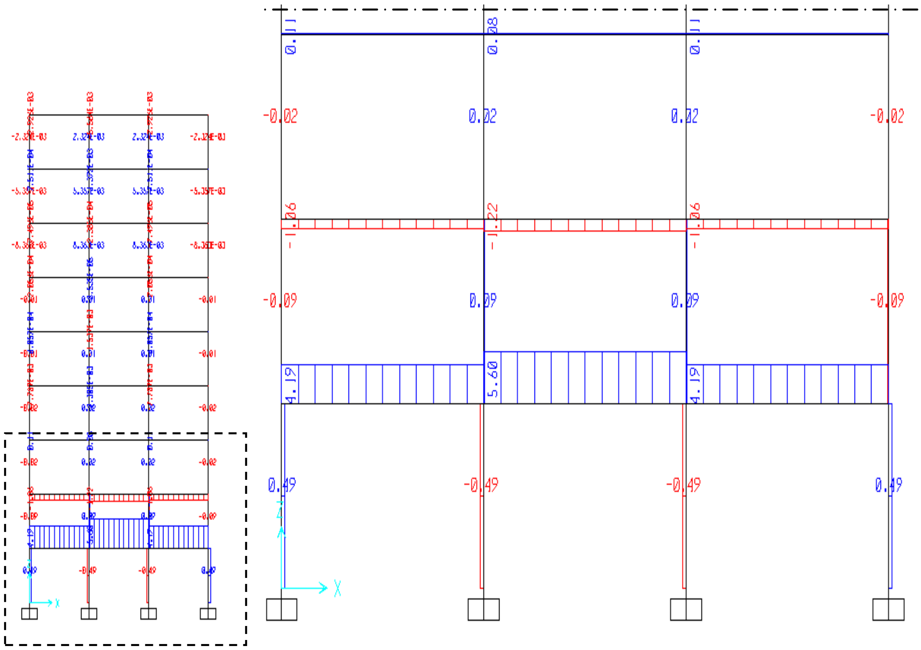


Fig. (3): Normal force diagrams

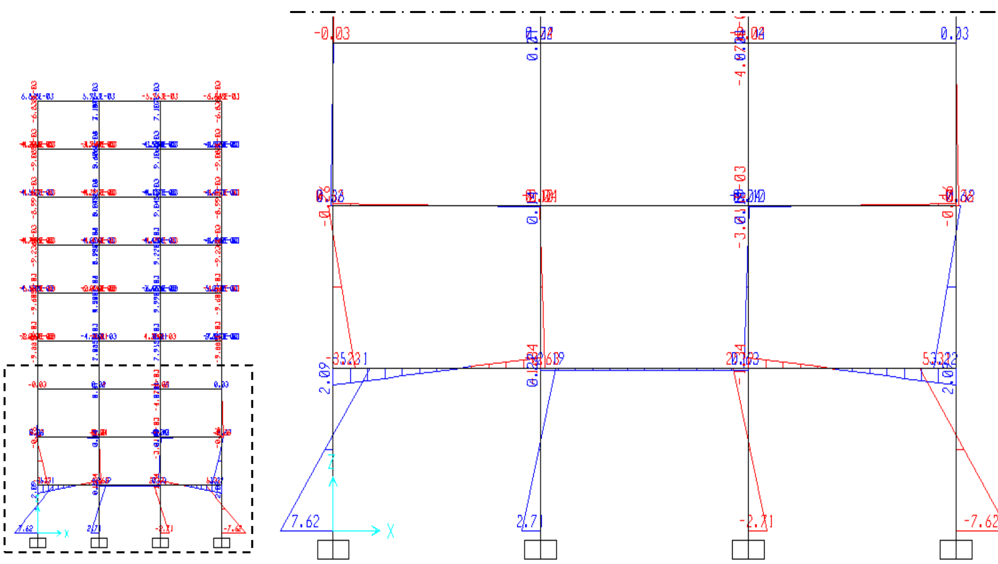


Fig. (4): Bending moment diagram (ton.m), case 2

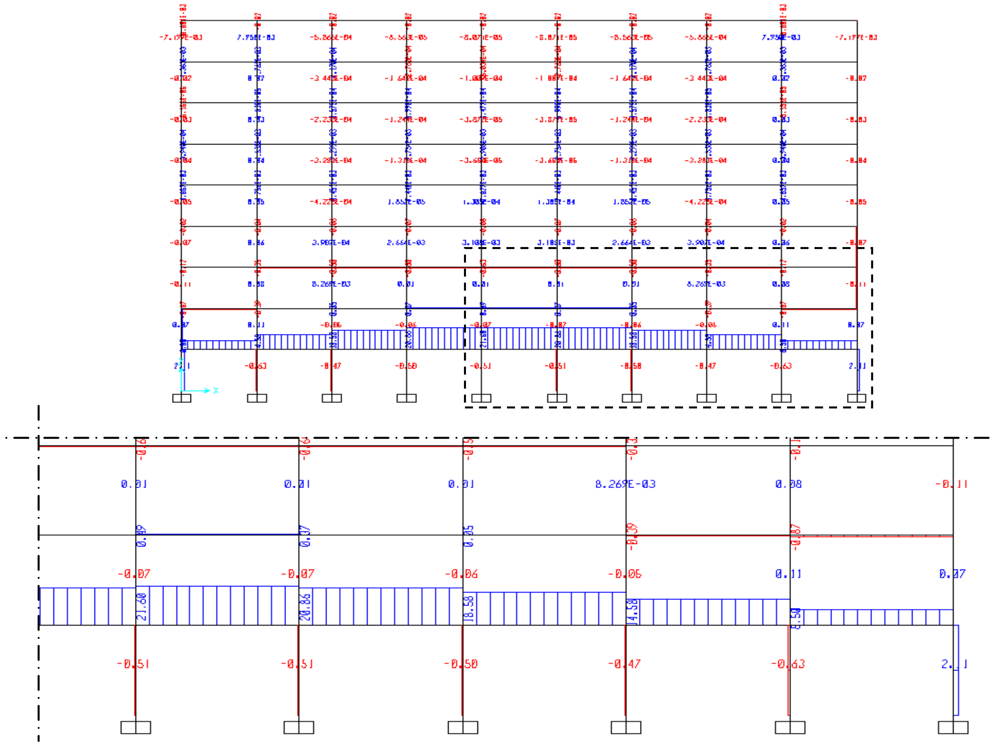


Fig. (5): Normal force diagram (ton), case 3

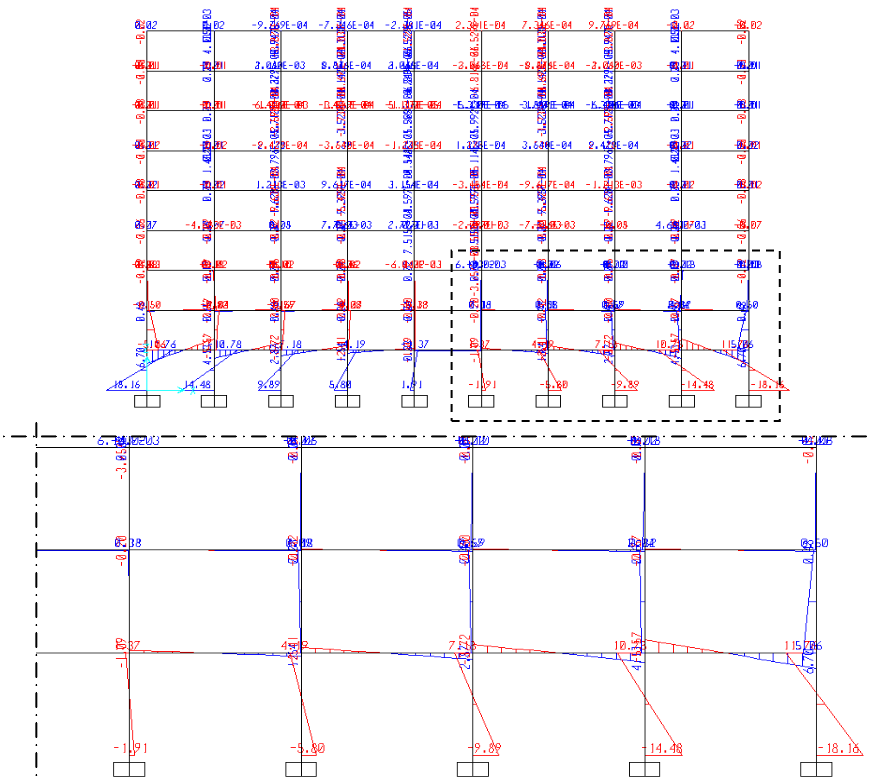
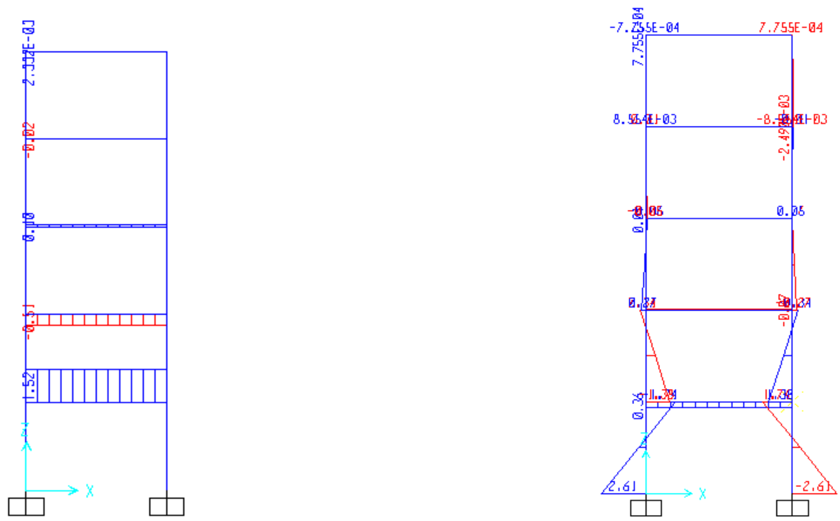
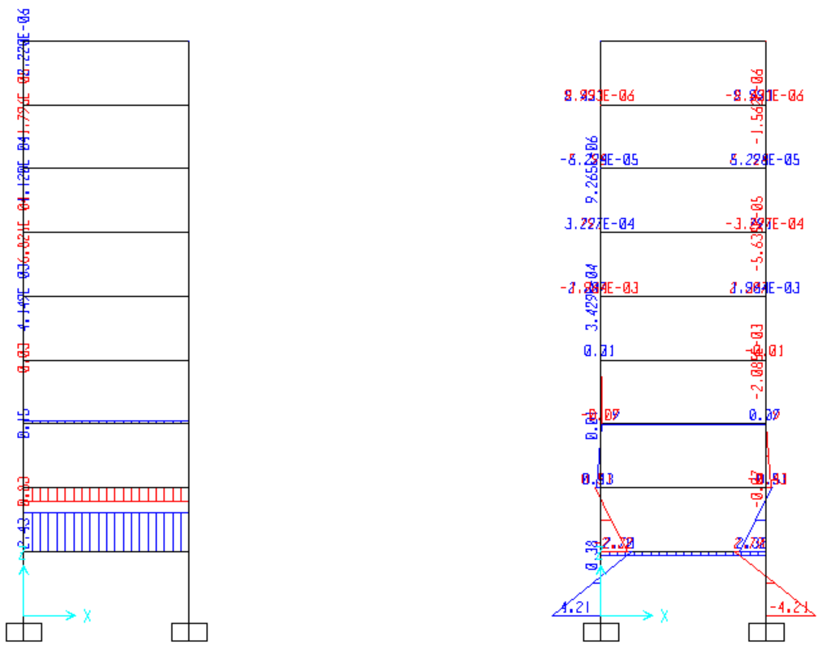


Fig. (6): Bending moment diagram (ton.m), case 3



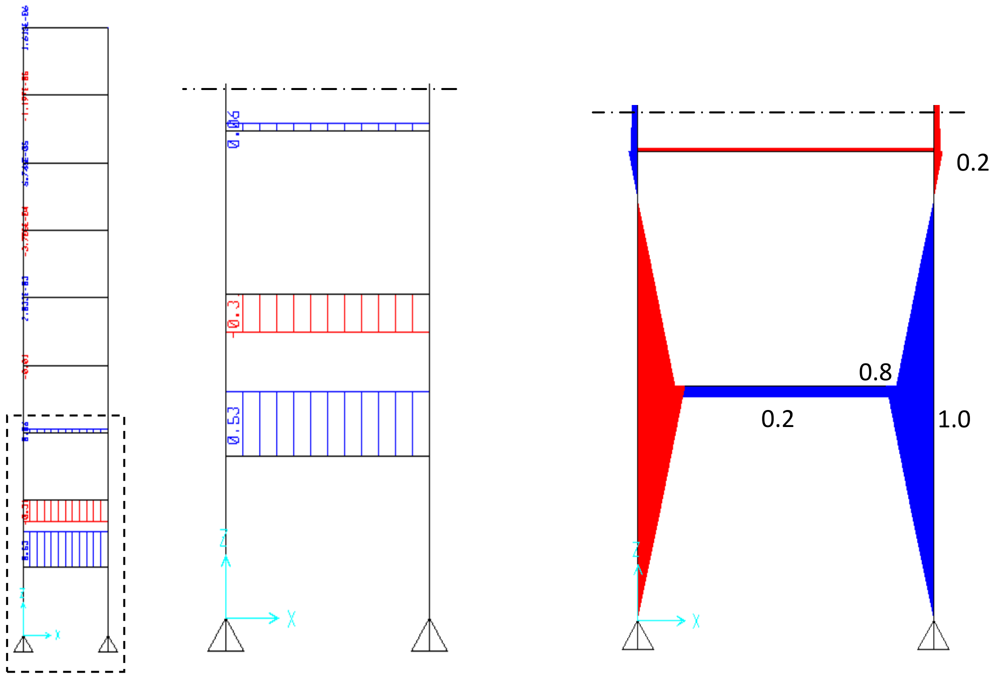
a: N.F.D (ton) b: B.M.D (ton.m)

Fig. (7): Normal force and bending moment diagrams, case 4



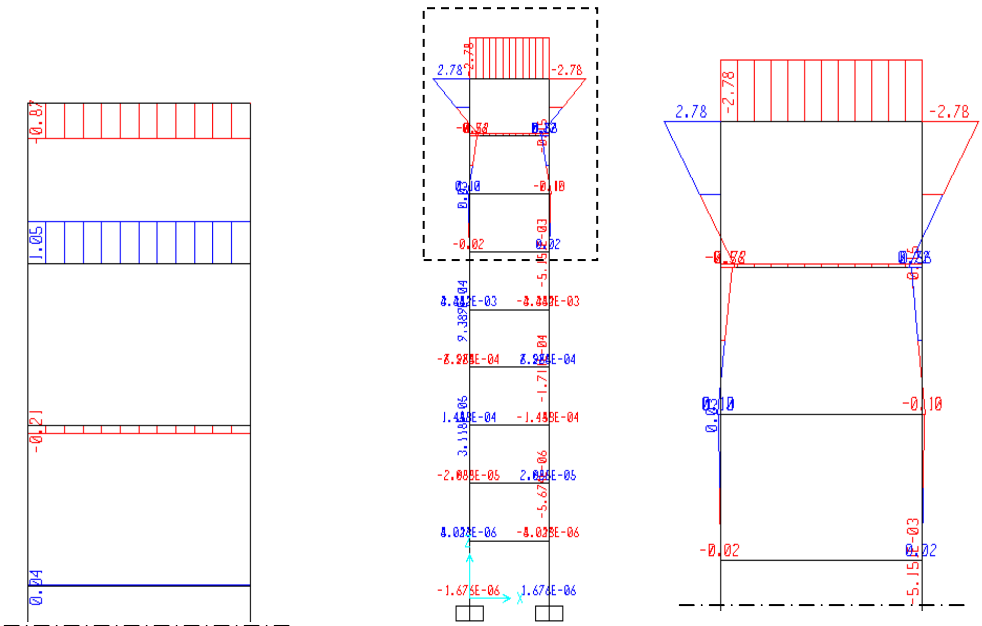
N.F.D (ton) B.M.D (ton.m)

Fig. (8): Normal force and bending moment diagrams, Case 5



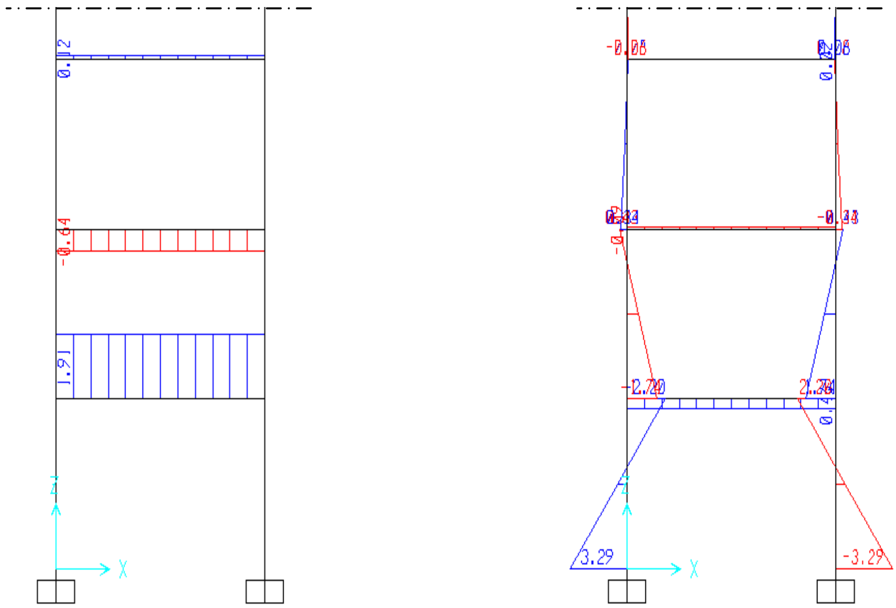
N.F.D (ton) B.M.D (ton.m)

Fig. (9): Normal force and bending moment diagrams, Case 6



N.F.D (ton) B.M.D (ton.m)

Fig. (10): Normal force and bending moment diagrams, Case 8



N.F.D (ton) B.M.D (ton.m)

Fig. (11): Normal force and bending moment diagrams, Case 9

3.2 Three-Dimension Analysis

To study the effect of temperature on three-dimension concrete building, the building shown in Figs. (12.a) and (12.b) was analyzed by SAP program. The building is 30.mx30.0 m dimension in plan; each side has five equal bays. The building contains a symmetrical core wall in the middle of 0.3 m thickness and dimensions 4.0x4.0 m in plan. All columns are 0.40x0.40 m cross section and all floors have a constant slab thickness of 0.2 m. The building has nine floors with a floor height of 4.0 m. All the concrete element of the building has a compressive strength of 250 kg/cm2 and a modulus of elasticity of 2.35x105 kg/cm2.

The building was analyzed twice, one to check the building when subjected to a uniform fall of temperature of 20 °C. The other is to check the roof slab when subjected to a linear gradient temperature with difference of 20 °C from outside to inside.

3.2.1 Results for Uniform Drop of Temperature

The results for uniform temperature drop are shown in Figs. from (13) to (17). Because of the symmetry, only the result for x direction is shown. In general, the results have the same trend as for 2-D calculations.

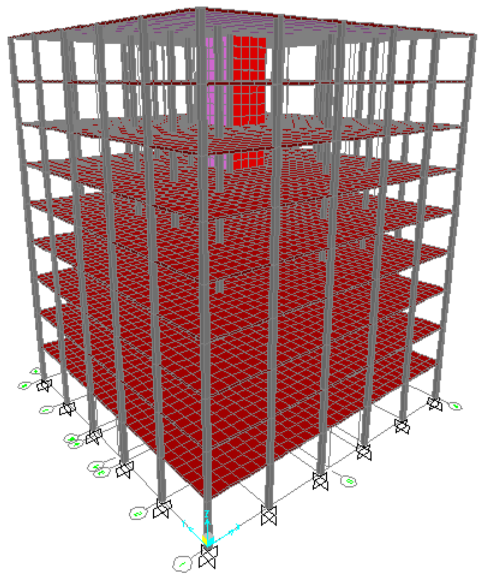


Fig. (12.a): Schematic 3-D view of the building

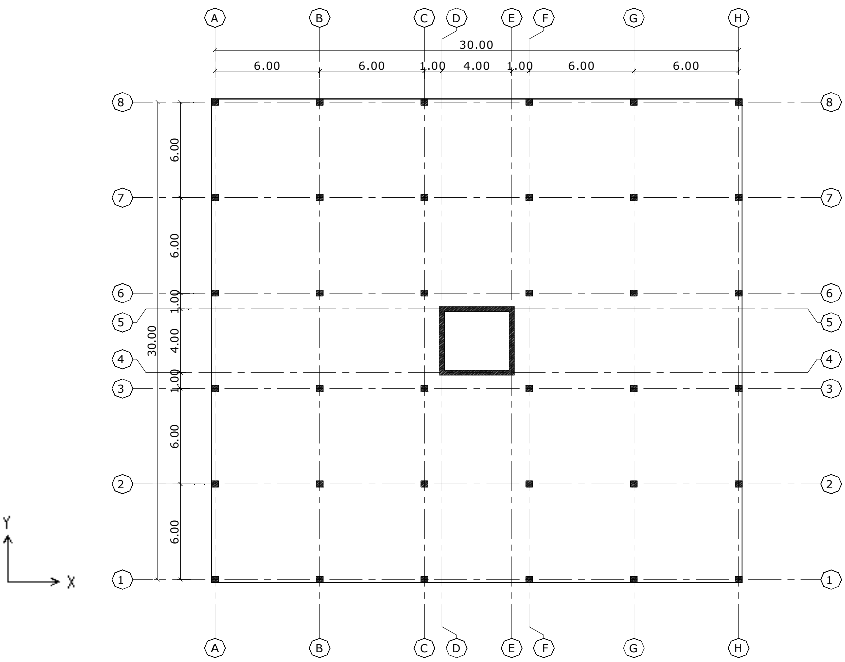


Fig. (12.b): Building floor plan

Figure (13) shows the contours of axial forces at the first floor slab. It is clear that this slab experienced the maximum axial forces. The axial force values were mostly positive with maximum values in the most inside panels. For the edge strep which locates at axe (A), the axial forces at the areas between columns were mostly positive except for few locations where the forces were negative. The maximum positive forces were experienced at the edge columns and close to the corners of the core wall that they were around the value of +1.9 ton/m. At the middle third of the building, the axial forces had an average value of +1.0 ton/m. The maximum negative forces were found close to the middle parts of the core wall that the value at some locations reached -0.43 ton/m.

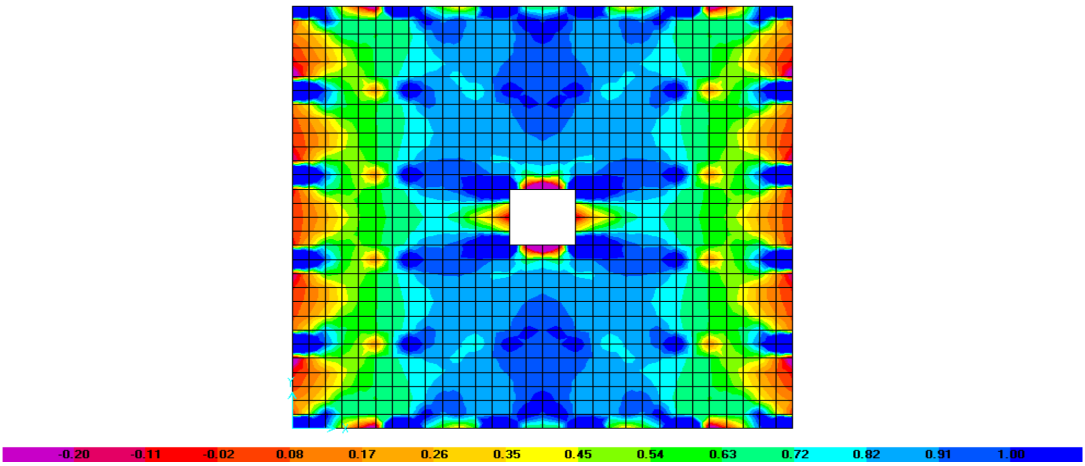


Fig. (13): Axial forces at 1st floor (ton/m)

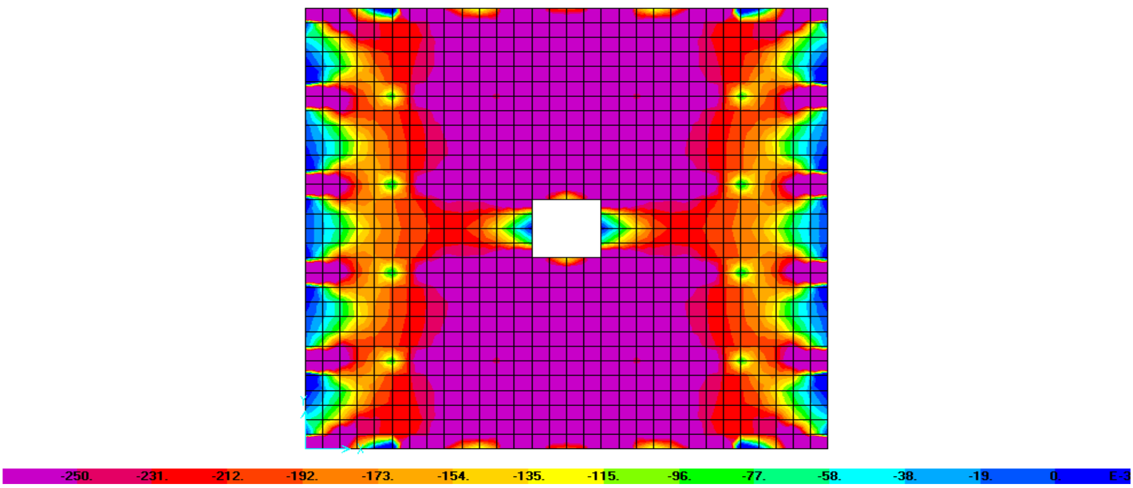


Fig. (14): Axial forces at 2nd floor (ton/m)

Figures (14), (15) and (16) show the results of axial forces at second, third and fifth floors. It can be seen that the sign of axial forces changed alternatively from positive to negative approximately with the same distribution trend as those of first floor. The values of forces decreased as the floor level increased that at the middle third of the slab, they were in average -0.25 kg/cm2 and +0.05 kg/cm2 for the second and third floors, respectively. At the fifth floor slab, the forces approximately diminished.

The maximum bending moment on the columns was experienced at the most external columns, with a value of 4.42 ton.m at the base, as shown in Fig. (17).

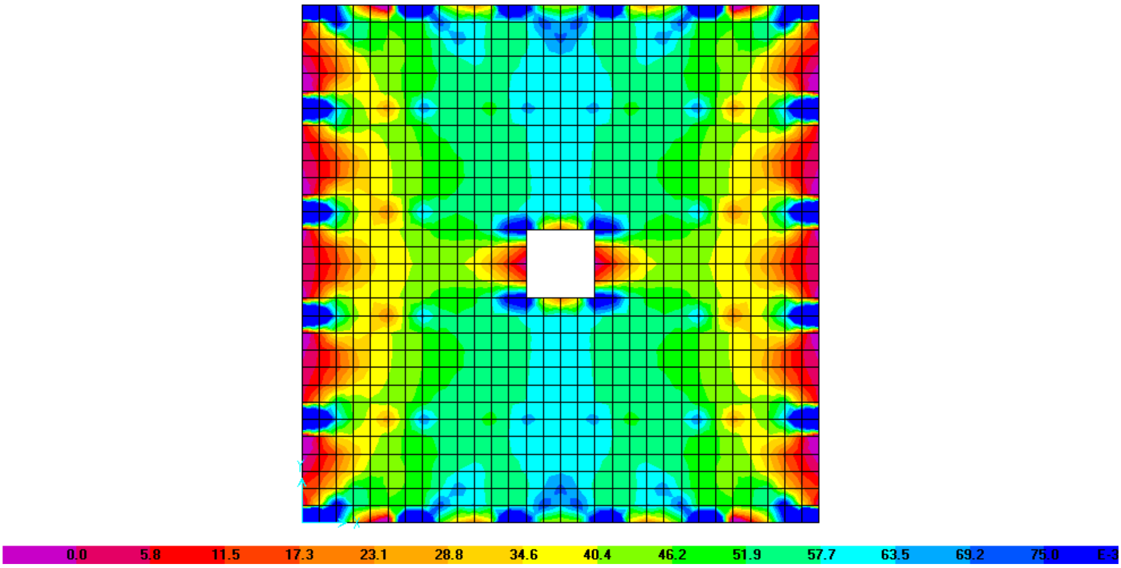


Fig. (15): Axial forces at 3rd floor x10^{-3}(ton/m)

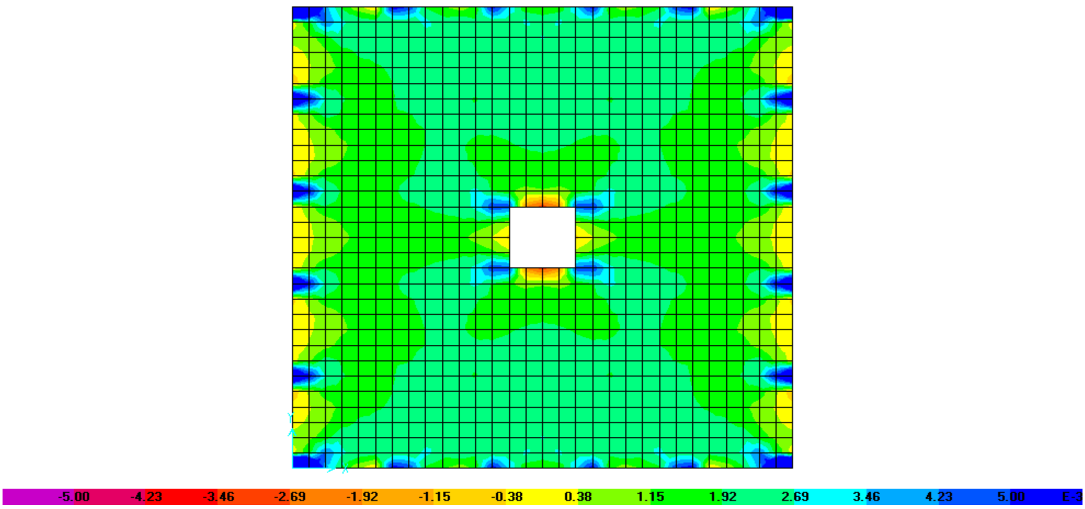
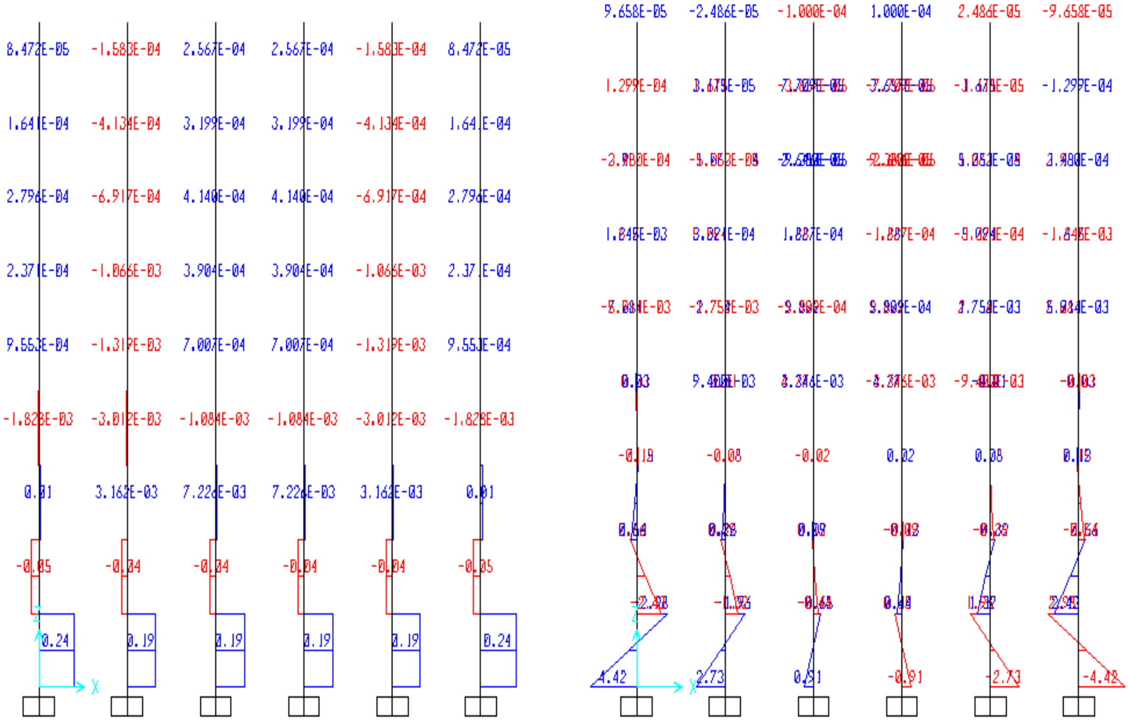
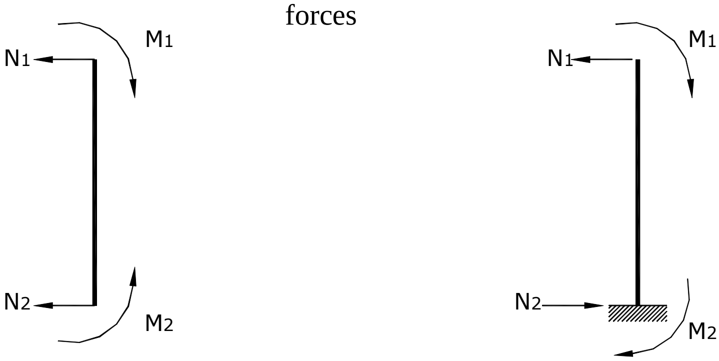


Fig. (16): Axial forces at 5th floor x10^{-3}(ton/m)



N.F.D (ton) B.M.D (ton.m)

Fig. (17): Axial and bending moment forces



a: Top and intermediate column b: Bottom end column

Fig. (18): Equilibrium of thermal forces

Now an important question is arising, why the forces are concentrated in these members which are close to the base connection?. A reasonable interpretation can be done by considering the equilibrium of thermal forces in columns illustrated in Fig. (18). If we consider the equilibrium of top and intermediate columns, as in Fig. (18-a), it can be seen that the column is subjected to forces N1 and N2. These forces happened due to the contraction or expansion of slab or beam which is attached to the column because of temperature change. Because the slab or the beam which attached at top and bottom of the column is exposed to the same temperature change, the forces N1 and N2 must have the same value. Because these forces have the same direction, only one condition can assure the equilibrium, N1 = N2 = 0.0. This means that for top and intermediate columns, no shear forces and consequently no bending moments will arise due to uniform temperature change. For a bottom end column, the equilibrium will be achieved by the reactions which will stem at the base. At this case, N2 is equal to N1 but in opposite direction. This interpretation imply an answer for another question, even though in case of roller-hinged frame is statically indeterminate, why it has zero forces?. Of course for roller-hinged frame N2 is equal to zero and no forces and moment will immerge.

In general, frame structures are characterized by their ability to undergo significant flexural deformation under these thermal loads. They are well known by the ability of their structural members to undergo rotation, such that the free thermal curvature change is not completely restrained. The thermal moments in the members are proportional to the degree of restraint. In addition to frames, slabs and walls fall into this category. As we moved away from the base end to upper floors, the degree of restraint rapidly decreases and very slight forces immerge according to the degree of restraint in the member.

The rotational feature above is of course automatically considered in a structural analysis using uncracked member properties. However, an additional reduction of the member thermal moments can occur if member cracking is taken into account.

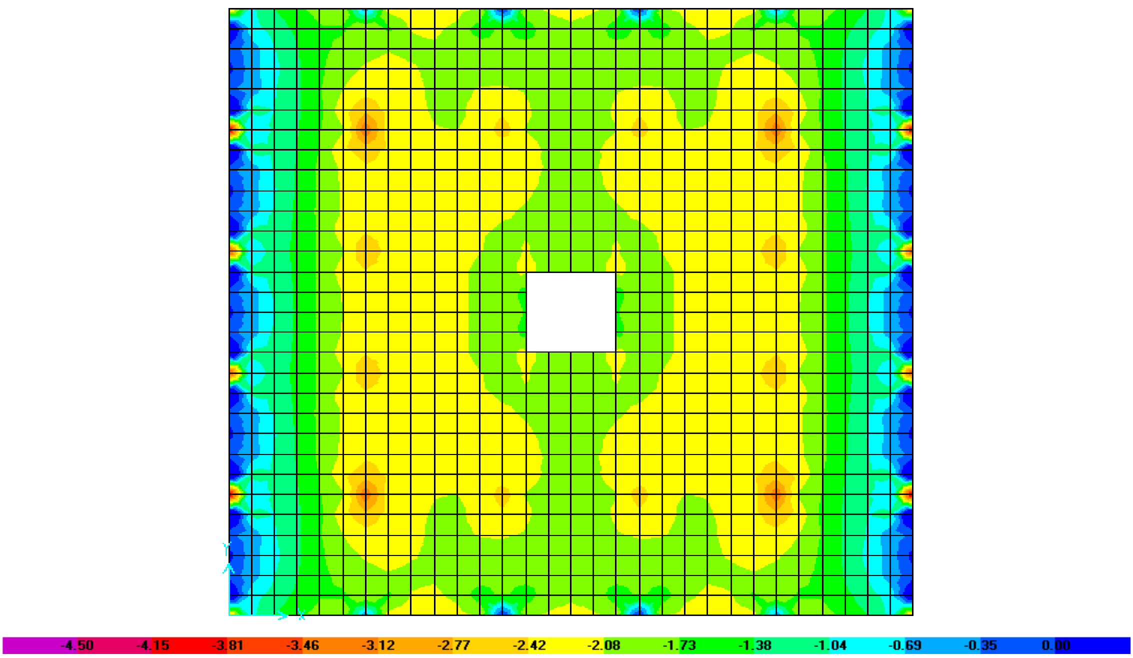


Fig. (19): Bending moments for roof slab due to linear gradient temperature (ton.m/m)

3.2.2 Results for Linear Gradient Temperature

The result for linear gradient temperature on the roof slab cross section is illustrated in Fig. (19). The result indicates that at the slab edges (axes A and H) and between columns, the bending moments produced tension at the bottom fiber having values ranging between 0.1 to 1.0 ton.m/m. By moving to inside, the bending moment changed it direction to produce tension at the top fiber with a peak value of 2.3 ton.m/m, approximately at a distance of 7 m from the edge. This can be clarified by a section at the middle of the slab as shown in Fig. (20). The maximum bending moment was experienced around column with an average value of 3.0 ton.m/m.

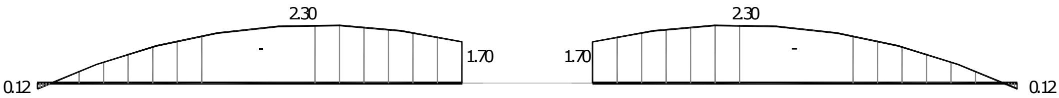


Fig. (20): Bending moment profile at section in the middle of the slab parallel to *x* axes (ton.m/m)

4. CONCLUSION

- A two and three-dimension analysis has been performed to investigate the temperature effect on multi-story concrete buildings. The structures were exposed to uniform drop and linear gradient temperature taking into consideration different cases of building configuration and support condition.
- Uniform rise or fall in temperature results in forces at the constraints. Generally, since the global constrains are limited to attachments at the foundations, the internal forces and stresses are concentrated in the elements close to the connection of frames and columns at these locations. The forces approximately diminish after the third floor.
- The number of floors has no effect on the resulted forces while higher values of forces are encountered by increasing the concrete strength, length and or the number of bays.
- For multiple bays or panels structure, the most affected elements by temperature are the most internal bay/panel and the most external columns.
- For frames with hinged connections to the foundations, the forces are drastically reduced.
- Even though multi-story frames with hinged-roller connections to the foundation are counted to be statically indeterminate structures, they experience no influence due to temperature changes.
- Linear gradient temperature on roof beams and slabs results in a significant bending moments in these beams and slabs.
- Daily and seasonal temperature changes have a significant influence in concrete structures. The resulted thermal forces should be taken into consideration when designing these structures.

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تأثير التغير في الحـرارة درجات على المنشآت الخرسانية متعددة الطوابق

هذا البحث يعرض دراسة تحليلية لمعرفة تأثير التغيرات اليومية والموسمية في درجات الحراة علي المنشآت الخرسانية متعددة الطوابق. تم عمل حسابات ثنائية وثلاثية الابعاد للمنشآت بتعريضها لاحمال حرارية مع الاخذ في الاعتبار حالات مختلفة لشكل المنشأ و كيفية الاتصال عند الاساسات. الاحمال الحرارية المطبقة كانت اما انخفاض منتظم في درجات الحرارة علي القطاع الخرساني أو تغيير تدريجي خطي في درجات الحرارة. تم مناقشة وتحليل النتائج ومقارنتها فيما بينها.