Seismic Analysis of Skew Shaped RC Building

Niravkumar G. Patel[1], Sunil M. Rangari[2]
Department of Civil Engineering
[1, 2] Savetribai Phule Pune University, Dr. D Y Patil School of Engineering, Lohegaon, Pune, Maharashtra 412105, India

Abstract- *The analysis and design of conventional multistoried RC buildings is now a routine practice. Any departure from much simplistic structural system needs an independent investigation for producing a rational design. The proposed work consists of a multistoried RC building with slanting vertical profile. Such structure can easily be tackled through the versatile finite element solution technique. Employing the same the following aspects have been covered.*

1. *Pseudo-static seismic analysis as per recommendations of IS 1893 (Part – I): 2002.*
2. *To understand the influence of damping ratio in above kind of analysis. The number of analysis are performed with damping ratios 2%, 5%, 10% and 20%. It has been observed that the normal practice of considering 5% damping ratio in case of concrete structure has validity.*
3. *The structural components are governed almost by the membrane actions and as the structure itself has a closed form cellular constitution displacements are quite small as is expected in such cases.*
4. *Marginal reinforcement is required to take care of membrane action in all plates of the structure. While designing the reinforcement the influence of flexural action is negligible.*

Keywords- Finite element analysis, Seismic analysis, Skew shaped building, Earthquake excitation, FORTRAN – 77.

I. INTRODUCTION

During an earthquake, failure of structures starts at the point of weakness. This weakness arises due to discontinuity in mass, stiffness and geometry of structure. The structures having this discontinuity are termed as Irregular structures. Irregular structures contribute a large portion of urban infrastructure. Vertical irregularities are one of the major reasons of failures of structures during earthquakes. For example structures with soft storey were the most notable structures that collapsed. So, the effect of vertically irregularities in the seismic performance of structures becomes really important. Height-wise changes in stiffness and mass render the dynamic characteristics of these buildings different from the regular building.

IS 1893 (Part – I): 2002 definition of Vertically Irregular structures:

The irregularity in the building structures may be due to irregular distributions in their mass, strength and stiffness along the height of building. When such buildings are constructed in high seismic zones, the analysis and design becomes more complicated. There are two types of irregularities:
1. Plan Irregularities
2. Vertical Irregularities

Plan Irregularities

i) Re-entrant Corners – Plan configurations of a structure and its lateral force resisting system contain re-entrant corners, where both projections of the structure beyond the re-entrant corner are greater than 15 percent of its plan dimension in the given direction.

ii) Out-of-Plane Offsets – Discontinuities in a lateral force resistance path, such as out-of-plane offsets of vertical elements.

iii) Non-parallel Systems – The vertical elements resisting the lateral force are not parallel to or symmetric about the major orthogonal axes or the lateral force resisting elements.

Vertical Irregularities

i) Stiffness Irregularity – A soft storey is one in which the lateral stiffness is less than 70 percent of the storey above or less than 80 percent of the average lateral stiffness of the three storey's above.

ii) Mass Irregularity – Mass irregularity shall be considered to exist where the seismic weight of any storey is more than 200 percent of that of its adjacent storey's. In case of roofs irregularity need not be considered.

iii) Vertical Geometric Irregularity – A structure is considered to be Vertical geometric irregular when the horizontal dimension of the lateral force resisting system in any storey is more than 150 percent of that in its adjacent storey.

iv) In Plane Discontinuity in Vertical Elements Resisting Lateral Force – An in plane offset of the lateral force resisting elements greater than the length of those elements.

v) Discontinuity in Capacity – Weak Storey – A weak storey is one in which the storey lateral strength is less than 80 percent of that in the storey above

II. OBJECTIVES OF THE STUDY

1. To study the response of skew shaped RC structure under seismic excitation.
2. Use of finite element method to obtain the response of skew shaped structure using FORTRAN – 77 compiler.
3. To obtain the displacements, stresses and flexural moments in members at various nodes and elements.
4. To check the suitability of Kirchhoff plate theory and Mindlin plate theory in the present study.
5. To check the manual calculations with the results obtained by using FORTRAN – 77, a small illustrative example is considered and then it is to be used to find response in the present study.

III. METHODOLOGY

The triangular plate element is denoted through its nodes numbered as (1-2-3) in an anti clockwise fashion. It is referred to local set of orthogonal axes (X, Y, Z) as shown in Fig. 1. In this, the plate element is confined to the (X, Y) plane and the z-axis is oriented in a direction normal to the plane of the plate.

A plate element displays three independent modes of deformation. These are membrane or plane stress mode, flexural mode and drilling mode.

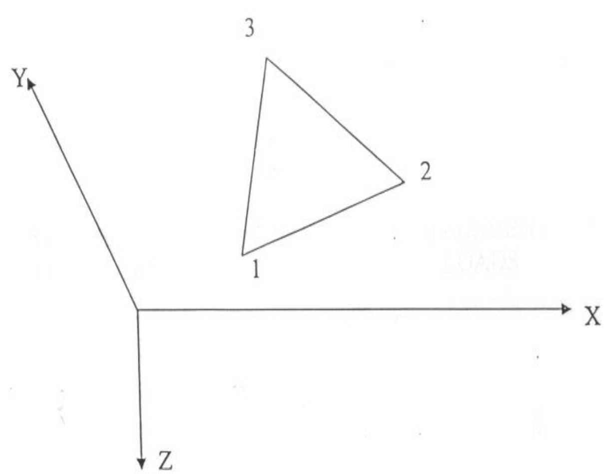


Fig. 1 Three nodded triangular plate element
(Local reference system)

a) Membrane action

1. Interpolation function

The displacement $[\delta_m]$ at a point (x, y) over the element is given by,

$$[\delta_m] = [N^m][\delta^m] \qquad \text{……(1)}$$

where, $[N^m]$ – matrix of the interpolation functions.
Now,

$$[\delta^m] = \begin{bmatrix} U \\ V \end{bmatrix} \qquad \text{……(2)}$$

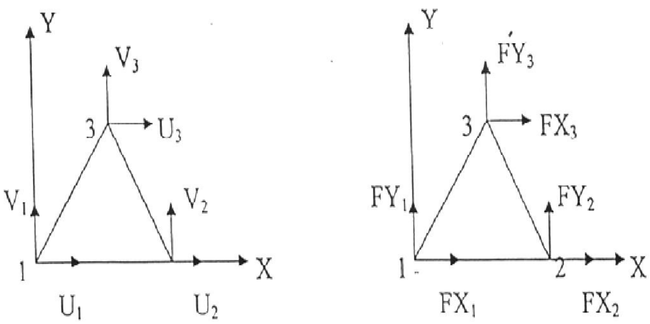


Fig. 2 (a) Element nodal displacements & (b) Nodal loads

$$[N^m] = \begin{bmatrix} N_1^m & 0 & N_2^m & 0 & N_3^m & 0 \\ 0 & N_1^m & 0 & N_2^m & 0 & N_3^m \end{bmatrix} \qquad \text{……(3)}$$

2. Strains

At a point (X, Y) the membrane action includes normal strain ε_x in X direction, normal strain ε_y in Y direction, and shear strain γ_{xy}. The strain components are denoted by element strain vector $[\varepsilon_m]$ defined as

$$[\varepsilon_m] = \begin{bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{bmatrix} \qquad \text{……(4)}$$

3. Stresses

Associated with the strains considered above a point (X, Y) over the element, the normal stress σ_X in X direction, normal stress σ_Y in Y direction and shear stress τ_{XY} are induced. The stress components that is denoted by an element stress vector $[\sigma_m]$ defined as

$$[\sigma_m] = \begin{bmatrix} \sigma_X \\ \sigma_Y \\ \tau_{XY} \end{bmatrix} \qquad \text{……(5)}$$

ISSN **[ONLINE]: 2395-1052**

4. Stiffness Matrix

Element stiffness matrix $[K_m]$ is given by,

$$[K_m] = t\iint [B_m]^T [C_m][B_m] dxdy \qquad \ldots\ldots(6)$$

where, t – element thickness. The integral is taken over the entire element area. In view of the fact that $[B_m]$ and $[C_m]$ are composed of constant coefficient,

$$[K_m] = t\Delta [B_m]^T [C_m][B_m] \qquad \ldots\ldots(7)$$

5. Element Load Vector

Element may be subjected to the pressure of intensities (P_X, P_Y) in (X, Y) directions and can be represented by a vector $[P_m]$ given by,

$$[P_m] = \begin{bmatrix} P_X \\ P_Y \end{bmatrix} \qquad \ldots\ldots(8)$$

b) Flexural action

Flexural action is carried out by the biaxial bending deformations. This kind of deformation involves at a point over the element, translation W in Z direction and rotation (θ_X, θ_Y) along (X, Y) axes, also force F_Z in Z direction and couples (C_X, C_Y) along (X, Y) axes. Consequently, nodal displacements and element nodal loads are as shown in Fig. 3(a) and Fig. 3(b) respectively. These are denoted through vectors $[\delta_f]$ and $[F_f]$ defined as

$$[\delta_f] = \begin{bmatrix} \delta_1^{\,f} \\ \delta_2^{\,f} \\ \delta_3^{\,f} \end{bmatrix} [F_f] = \begin{bmatrix} F_1^{\,f} \\ F_2^{\,f} \\ F_3^{\,f} \end{bmatrix} \qquad \ldots\ldots(9)$$

The sub vectors ($\delta_1^{\,f}$, $F_1^{\,f}$) etc are as below

$$[\delta_1^{\,f}] = \begin{bmatrix} W_1 \\ \theta_{X1} \\ \theta_{Y1} \end{bmatrix} [F_1^{\,f}] = \begin{bmatrix} F_{Z1} \\ C_{X1} \\ C_{Y1} \end{bmatrix} \qquad \ldots\ldots(10)$$

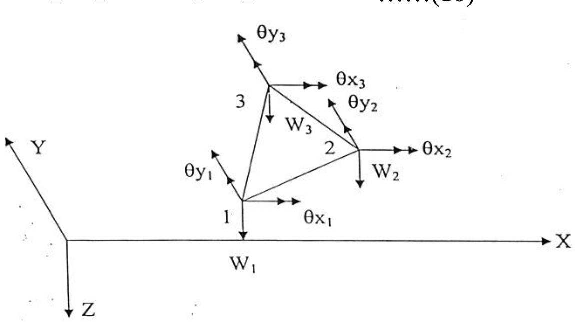


Fig. 3(a) Element nodal displacements

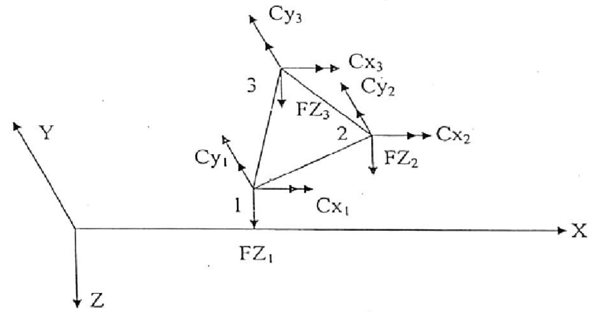


Fig. 3(b) Element nodal loads

1. Interpolation function

W is the deflection at point (X, Y) over the element and is given by,

$$W = [N^f][\delta^f] \qquad \ldots\ldots(11)$$

where, $[N^f]$ – matrix of interpolation functions.

Assuming cubic variation of W over the element domain, subject to condition that θ_X = -dW/dY and θ_Y = dW/dX, it could show that,

$$[N^f] = \begin{bmatrix} N_1^f & N_{1X}^{\,f} & N_{1Y}^{\,f} & N_2^f & N_{2X}^{\,f} & N_{2Y}^{\,f} & N_3^f & N_{3X}^{\,f} & N_{3Y}^{\,f} \end{bmatrix} \quad \ldots..(12)$$

2. Generalized Strains

Consider a slice at a distance Z from the neutral surface of the plate. The slice is subjected to normal strains (ε_X, ε_y) and shear strains γ_{XY}. The strain in the element is given by

$$[\varepsilon_r] = \begin{bmatrix} -\partial^2 W/\partial X^2 \\ -\partial^2 W/\partial Y^2 \\ 2\partial^2 W/\partial X\partial Y \end{bmatrix} \qquad \ldots\ldots(13)$$

where, $-\partial^2 W/\partial X^2$ and $-\partial^2 W/\partial Y^2$ are the curvatures around Y axis and X axis respectively and $2\partial^2 W/\partial X\partial Y$ represents the twist of the deformed surface.

3. Generalized Stresses

At a point over a slice at a distance Z from the neutral surface of plate, there prevails normal stress (σ_X, σ_Y) and shear stress τ_{XY}. The stresses in the element is given by,

$$[\sigma_f] = \begin{bmatrix} M_X \\ M_Y \\ M_{XY} \end{bmatrix} \qquad \ldots\ldots(14)$$

herein, M_X and M_Y are the bending moments in X and Y directions i.e. around Y and X axis respectively, whereas M_{XY} is the twisting moment.

4. Stiffness matrix

Element stiffness matrix $[K_f]$ is given by,

$$[K_f] = \iint [B_f]^T [C_f][B_f] dxdy \qquad \text{……(15)}$$

The integral to be taken over the element area and once again three-point numerical integration is convenient for the purpose.

5. Element load vector

The element may be subjected to the pressure of intensities P_Z acting along Z directions and corresponding $[F_f]$ turns out to be,

$$[F_f] = \iint P_Z [N^f]^T dxdy \qquad \text{……(16)}$$

The integral is evaluated over the element surface through three-point integration scheme.

c) Drilling action

Drilling action is characterized by rotation θ_Z and couple C_Z around Z axis. Consequently, the element nodal displacements and the element nodal loads are shown in Fig. 4(a) and Fig. 4(b) respectively. These are denoted through vectors $[\delta_d]$ and $[F_d]$ defined as

$$[\delta_d] = \begin{bmatrix} \theta_{Z1} \\ \theta_{Z2} \\ \theta_{Z3} \end{bmatrix} [F_d] = \begin{bmatrix} C_{Z1} \\ C_{Z2} \\ C_{Z3} \end{bmatrix} \qquad \text{……(17)}$$

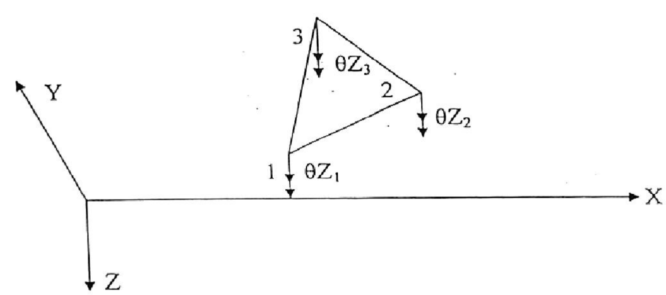


Fig. 4(a) Element nodal displacements

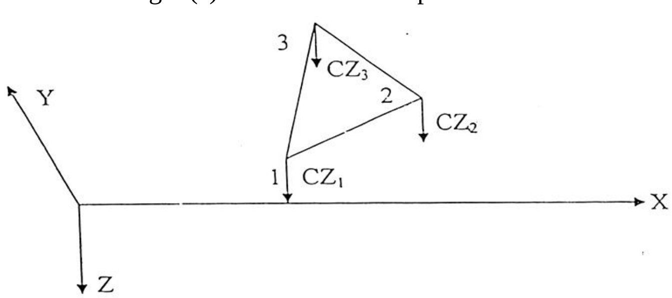


Fig. 4(b) Element nodal loads

Element local equilibrium is then converted to element global equilibrium using transformation matrix.

d) Transformations

The element with local axes (x, y, z) is placed in the global (X, Y, Z) spaces. A vector P in these spaces will have components (P_x, P_y, P_z) in (x, y, z) directions and components (P_X, P_Y, P_Z) in (X, Y, Z) directions.

It follows from vector mechanism that,

$$\begin{bmatrix} P_Y \\ P_Z \\ P_X \end{bmatrix} = [T] \begin{bmatrix} P_x \\ P_y \\ P_z \end{bmatrix} \qquad \text{……(18)}$$

where, [T] – mechanical transformation matrix defined as,

$$[T] = \begin{bmatrix} \lambda Xx & \lambda Xy & \lambda Xz \\ \lambda Yx & \lambda Yy & \lambda Yz \\ \lambda Zx & \lambda Zy & \lambda Zz \end{bmatrix} \qquad \text{……(19)}$$

It could be shown that,

$$[F_e] = [R][F_e{}^L]; \qquad [K_e] = [R]^T [K_e{}^L][R] \qquad \text{……(20)}$$

where, [R] – rotation matrix defined by the diagonal placement of [T] matrix as below

$$[R] = \begin{bmatrix} [T] & & & & & \\ & [T] & & & & \\ & & [T] & & & \\ & & & [T] & & \\ & & & & [T] & \\ & & & & & [T] \end{bmatrix} \qquad \text{……(21)}$$

IV. RESULTS

The structure being considered for study is a skew shaped reinforced concrete seven storey building with all vertical walls inclined as shown in figure below.

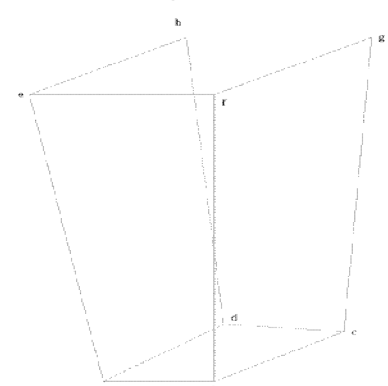


Fig. 5 Three – dimensional view of the structure

The idealization of the structure is shown in Appendix – I.

The solution by finite element technique for evaluation of Displacements, Member Stresses and Bending developed in the component can be carried out through three basic steps:

1. Finite element idealization of the structure being analyzed.
2. Formulation and solution of the equations governing equilibrium of the idealized system and
3. Evaluation of the structural response.

The results for the skew shaped building under seismic condition are obtained in the form of displacements, stresses and moments in the members for Zone – IV as per IS 1893 – 2002. The results are presented in the form of table, figures and contours.

1. Nodal displacements

The displacements developed due to horizontal seismic forces are function of damping co-efficient ('ξ'). Following the spectrum curves of IS 1893: 2002, the information being available for 2%, 5%, 10% and 20% the analysis was performed for consideration being given to each of the damping co-efficient. Force being applied in x – direction the displacement developed is negligible in y – direction but is significant in x – direction as well as in z – direction.

The displacements at various floor levels are presented in form of contours and presented in Appendix –II.

The maximum horizontal displacements at various floor levels for various damping coefficients are given below.

Table 1: Maximum displacements at various floor levels

Damping Ratio 'ξ'	Z = 3m	Z = 6m	Z = 9m	Z = 12m	Z = 15m	Z = 18m	Z = 21m
2 %	0.5	1.20	1.92	2.70	3.47	4.20	4.90
5 %	0.4	0.83	1.34	1.87	2.39	2.89	3.39
10 %	0.3	0.64	1.02	1.42	1.82	2.20	2.56
20 %	0.2	0.47	0.78	1.07	1.37	1.67	1.95

Scale = Value x 10^{-2}mm

From above Table it shows that increasing the damping ratio from 2% to 5% the horizontal displacement in X – direction reduces significantly. However, for damping ratio varying from 5% to 20% the decrease in displacements in x- direction is marginal. In general, it is a practice that for variety of concrete structures consideration of 5% damping ratio in analysis is adequate. Hence in subsequent details regarding stresses and moments developed, the results in respect to only 5% damping is presented.

Though the displacements are largest for damping ratio 2% their magnitude appear to be quite small. This is expected because the structural system of the building is a closed box cellular in which the dominant action is only membrane hence the smaller values of the displacements.

2. Member Stresses

The membrane stresses at various floor levels are essentially due to influence of in plane displacements. The contours of the basic membrane stress components σ_x, σ_y, τ_{xy} in case of various floors are obtained. σ_x and σ_y are normal stresses in X and Y direction respectively and τ_{xy} is shear stress. The stresses obtained for slab elements in form of contours at various floor levels are presented in Appendix - III and maximum stresses at various floors are shown in Table 2.

Appendix – III shows that the normal stresses induced in the elements are of same order in X and Y – directions in increasing pattern. The same increasing pattern is observed in shear stresses

Table 2: Maximum stresses at various floor levels

Member Stresses (kN/m^2)	Z = 3m	Z = 6m	Z = 9m	Z = 12m	Z = 15m	Z = 18m	Z = 21m
σ_x & σ_y	80	80	60	50	40	40	40
τ_{xy}	50	50	40	40	20	10	10

For elements at all levels the membrane normal stresses are maximum at lowest level and with increase in height it reduces. Same observation is made with respect to shear stresses. In view of this observation it is obvious that some reinforcement would be required whose amount will depend on values of above stresses. It is a general practice to provide required steel in manner of equal steel area at top and bottom section of the plates. It is obvious that such reinforcement would be needed in both x and y – direction. The shear stress would be accounted through diagonal placement of reinforcement at the corner of the plates.

It is observed from Table 2 that the normal stresses does not changes up to second floor and thereafter it reduces significantly due to reduction in the base shear in subsequent floors. The same observation is made for shear stresses. The normal stresses reduce by 50% between first floor and top floor and shear stress by 80%.

3. Flexural Moments

The membrane bending at various floor levels are essentially due to influence of in plane displacements. The contour of the basic bending moments M_x, M_y and M_{xy} in case of various floors are presented in Appendix – IV. M_x and M_y are moments due to in plane bending and M_{xy} is moment due to rotation.

Table 3: Maximum flexural moments at various floor levels (Positive moments)

Flexural Moments (kNm/m run)	Z = 3m	Z = 6m	Z = 9m	Z = 12m	Z = 15m	Z = 18m	Z = 21m
M_x& M_y	4	4	4	6	5	10	10
M_{xy}	2	2	2	3	3	3	4

Table 4: Maximum flexural moments at various floor levels (Negative moments)

Flexural Moments (kNm/m run)	Z = 3m	Z = 6m	Z = 9m	Z = 12m	Z = 15m	Z = 18m	Z = 21m
M_x& M_y	-6	-4	-4	-4	-5	-3	-10
M_{xy}	-2	-2	-2	-2	-1	-3	-4

From the details presented in Table 3 and 4 the following conclusions are noteworthy:

a) At each floor level the positive as well as negative moments get developed. However, their values are more or less comparable. In view of this the additional reinforcement requirement could be computed ignoring the sign of the moment because as mentioned earlier provision of equal steel on both the faces the lever arm between two levels of reinforcement would decide the additional steel area required.

b) The value of M_{xy} is normally not important in further enhancing reinforcement requirements. In fact with equal steel being provided on either face the M_{xy} is just added to M_x and M_y to update the values of M_x and M_y.

The structure under study was also analyzed in Staad Pro. and the results obtained are comparable as shown below.

Table 5: Maximum horizontal displacements at various floor levels (as obtained from Staad Pro.)

Damping Ratio 'ξ'	Z = 3m	Z = 6m	Z = 9m	Z = 12m	Z = 15m	Z = 18m	Z = 21m
5 %	0.22	0.57	0.98	1.44	1.9	2.36	2.8

Scale = Value x 10^{-2}

Table 6: Maximum stresses at various floor levels

Member Stresses (kN/m²)	Z = 3m	Z = 6m	Z = 9m	Z = 12m	Z = 15m	Z = 18m	Z = 21m
σ_x& σ_y	90	78	58	38	40	12	13.7

Table 7: Maximum moments at various floor levels (Positive moments)

Flexural Moments (kNm/m run)	Z = 3m	Z = 6m	Z = 9m	Z = 12m	Z = 15m	Z = 18m	Z = 21m
M_x& M_y	6.6	3.4	4.5	6	5.3	10	10

V. CONCLUSION

On the basis of details presented above it is possible to conclude as follows:

1. Consideration of 5% damping ratio is adequate from practical point of view.
2. The membrane stresses at all floor levels are tensile in nature.
3. The flexural moments developed have both positive as well as negative moments.
4. As per walls are concerned flexural moments are negligible where as membrane stresses for small region at the base of the wall is compressive whereas above the base membrane stresses are tensile with maximum values being in the top region.
5. The structure studied in FORTRAN – 77 is also analyzed in Staad Pro. and the displacement results obtained are comparable.

REFERENCES

[1] PICA, R. D. WOOD and E. HINTON, "Finite Element Analysis of Geometrically Nonlinear Plate Behaviour using a Mindlin Formulation", Computers and Structures, Vol. 11, pp. 203 – 215 (1980).

[2] B.G. Birajdar and S.S. Nalawade, “Seismic Analysis of Buildings resting on sloping ground”, The Proceedings of 13th World Conference on Earthquake Engineering, Canada, Paper no. 1472 (2004).

[3] Clough R. W. and K. L. Wilson, “Inelastic Earthquake Response of Tall Building”, The Proceedings of 3rd World Conference on Earthquake Engineering, New Zealand, Vol. II Section II, pp. 68 – 89 (1965).

[4] D. W. Murray and E. L. Wilson, “Finite element large deflection analysis of plates”, A.S.C.E, pp. 143-165 (1969).

[5] E. D. L. Pugh, E. Hinton and O. C. Zienkiewicz, “A study of quadrilateral plate bending elements with reducedintegration”, International Journal of Numerical Mathematical Engineering, Vol. 12, pp. 1059-79 (1978).

[6] E. Hinton and N. Bicanic, “A comparison of Lagrangian and Serendipity Mindlin plate elements for free vibration”, Journal of Computers and Structures, Vol. 10(3), pp. 483 – 493 (1979).

[7] F. Mazzaand M. Mazza, “A Numerical Model for the nonlinear Seismic Analysis of Three – Dimensional R. C. Frame”, The Proceedings of 14th World Conference on Earthquake Engineering, Beijing, China (2008).

[8] Hajime Umemura, Hiroyuki Aoyama and Haruo Takizawa, “Analysis of the behavior of Reinforced Concrete Structures during Strong Earthquake based on Empirical Estimation of Inelastic Restoring Force Characteristics of Members”, WCEE, Vol. 2, Article no. 2201 (1972).

[9] Okamoto S., S. Nakata, Y. Kitagawa, M. Yoshimura and T. Kaminosono, “A Progress Report on the Full-scale Seismic Experiment of a Seven-story Reinforced Concrete Building – Part of the U.S. – Japan Cooperative Program”, BRI Research Paper No. 94, Building Research Institute, Ministry of Construction (1982).

[10] Örvar Jónsson, “The dynamic behaviour of multi-story reinforced concrete building in a seismic and windy environment”, MSc. Thesis in Civil Engineering specialization Structural Engineering (2014).

[11] Otani S., “Nonlinear Dynamic Analysis of Reinforced Concrete Building Structures”, Canadian Journal of Civil Engineering, Vol. 7, No. 2, pp. 333 – 344 (1980).

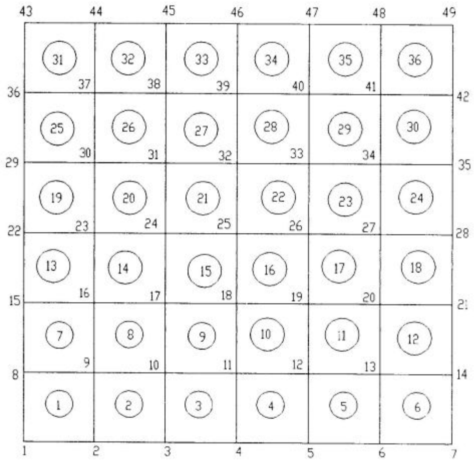
[12] Ray W. Clough and James L. Tocher, “Finite Element Stiffness Matrices for Analysis of Plate Bending”, Journal of Aeronautical Sciences, pp. 515 – 545 (1965).

[13] Thomas J. R. Hughes, Martin Cohen and Medhat Haroun, “Reduced and Selective Integration Technique in the Finite Element Analysis of Plates”, Journal of Nuclear Engineering and Design, Special Issue Structural Mechanics, Vol. 46(1), pp. 203 – 222 (1978).

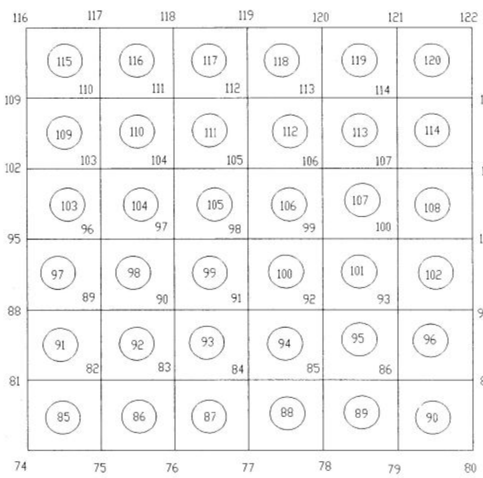
[14] T. J. R. Hughes, R. L. Taylor and W. Kanoknukulchai, “A simple and efficient finite element for plate bending”, International Journal of Numerical Mathematical Engineering, pp. 1529-43 (1977).

[15] Toshimi KABEYASAWA, Hitoshi SHIOHARA, Shunsuke OTANI and Hiroyuki AOYAMA, “Analysis of the Full-scale Seven-story Reinforced Concrete Test Structure“, Journal (B), The Faculty of Engineering, University of Tokyo, Vol. XXXVII, No. 2, pp. 432 – 478 (1983).

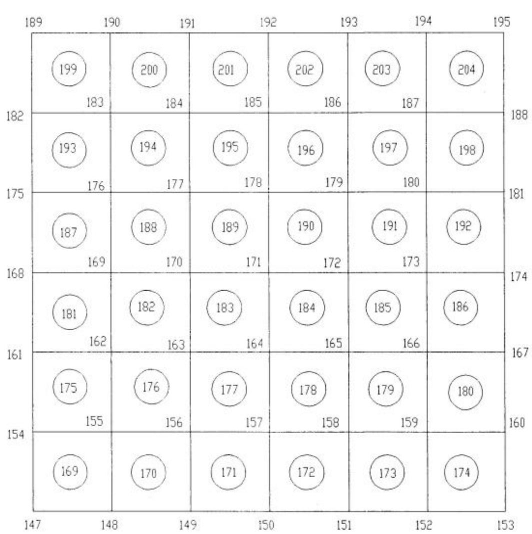
APPENDIX – I



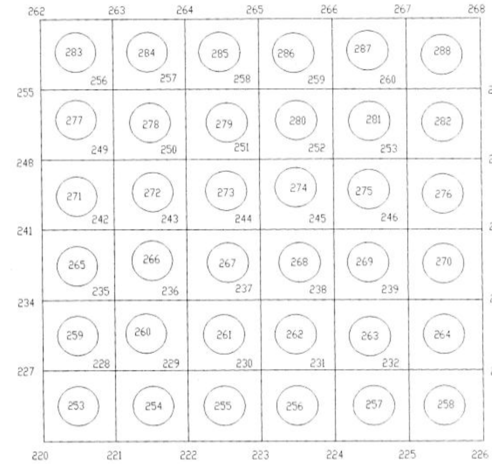
(a) Floor slab at Z = 0.0 m & X = Y = 6 m
Floor idealization for base floor



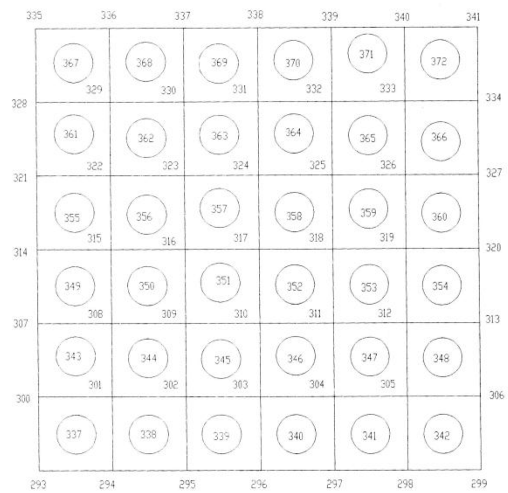
(b) Floor slab at Z = 3.0 m & X = Y = 6.5 m
Floor idealization for 1st floor



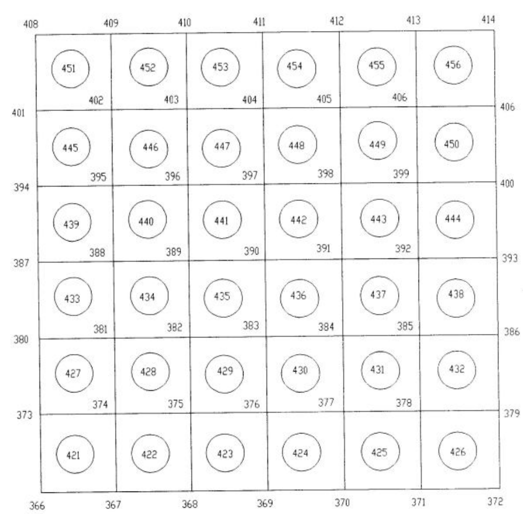
(c) Floor slab at Z = 6.0 m & X = Y = 7 m
Floor idealization for 2nd floor



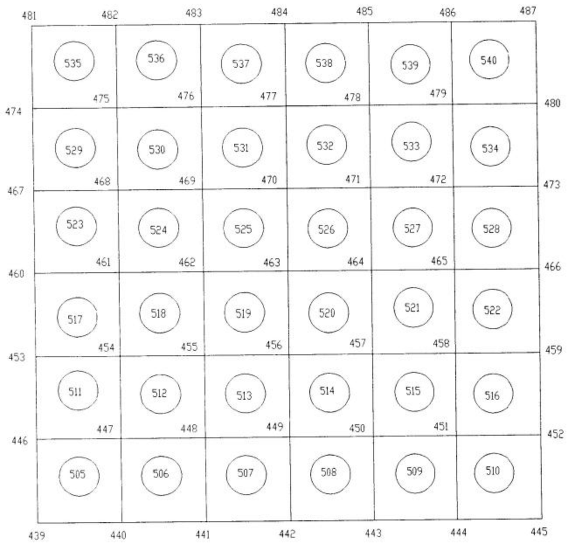
(d) Floor slab at Z = 9.0 m & X = Y = 7.4 m
Floor idealization for 3rd floor



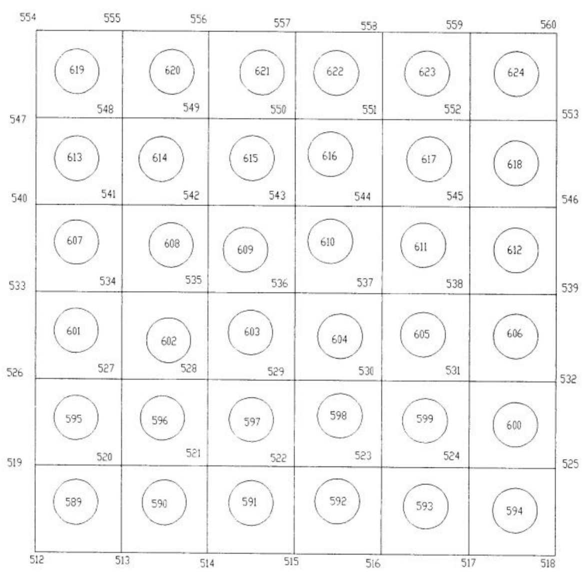
(e) Floor slab at Z = 12.0 m & X = Y = 7.8m
Floor idealization for 4th floor



(f) Floor slab at Z = 15.0 m & X = Y = 8.2 m
Floor idealization for 5th floor



(g) Floor slab at Z = 18.0 m & X = Y = 8.6 m
Floor idealization for 6th floor

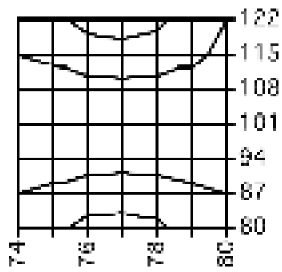


(h) Floor slab at Z = 21.0 m & X = Y = 9 m
Floor idealization for 7th floor

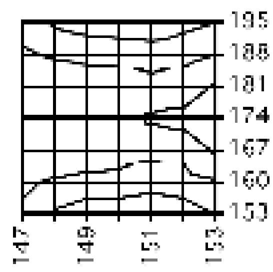
APPENDIX – II

Displacement contour at various floor levels

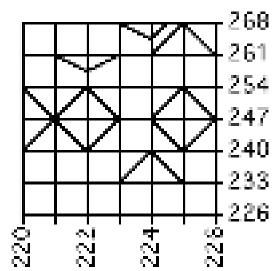
Displacements 'u' at Z=3m



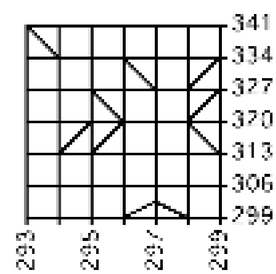
Displacements 'u' at Z=6m



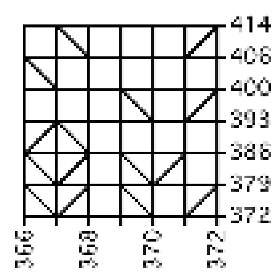
Dsiplacements 'u' at Z=9m



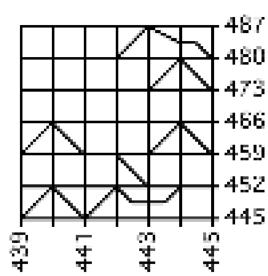
Displacements 'u' at Z=12m



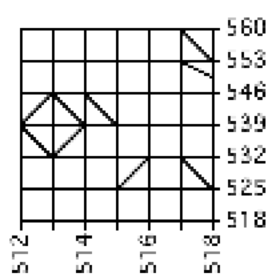
Displacements 'u' at Z=15m



Displacements 'u' at Z=18m

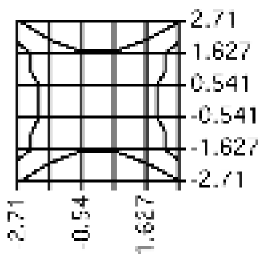


Displacements 'u' at Z=21m

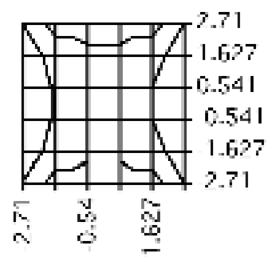


APPENDIX – III
Member stresses at various floor levels

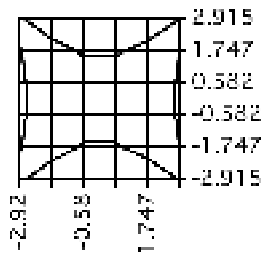
Member Stress 'σx' at Z=3m



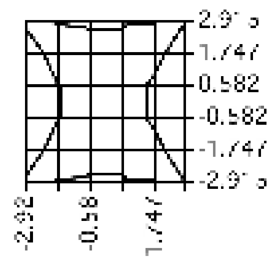
Member Stress 'σy' at Z=3m



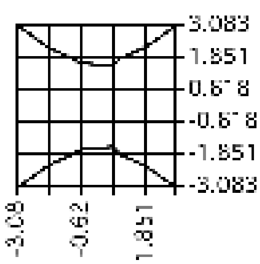
Member Stress 'σx' at Z=6m



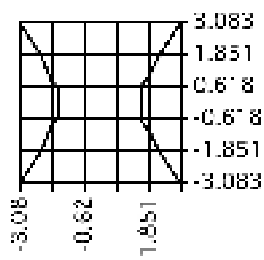
Member Stress 'σy' at Z=6m



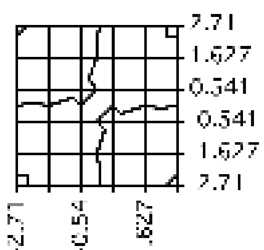
Member Stress 'σx' at Z=9m



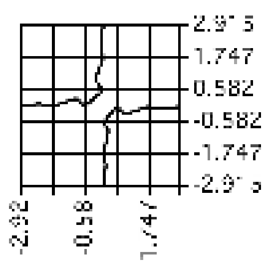
Member Stress 'σy' at Z=9m



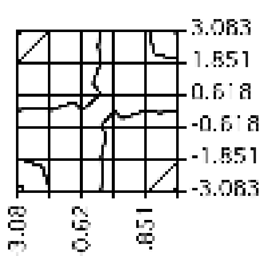
Member Stress 'τxy' at Z=3m



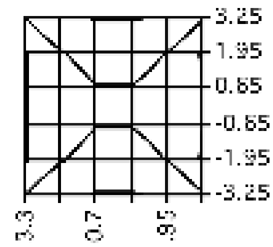
Member Stress 'τxy' at Z=6m



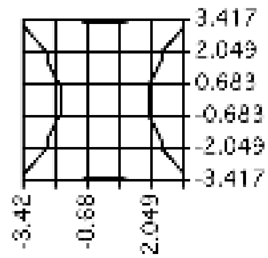
Member Stress 'τxy' at Z=9m



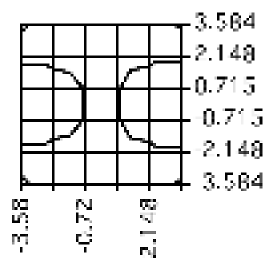
Member Stress 'σx' at Z=12m



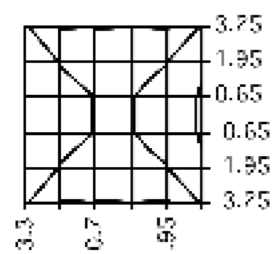
Member Stress 'σx' at Z=15m



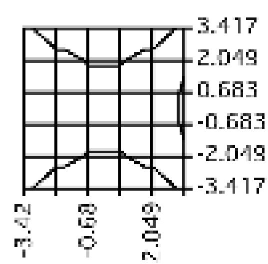
Member Stress 'σx' at Z=18m



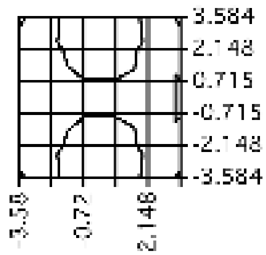
Member Stress 'σy' at Z=12m



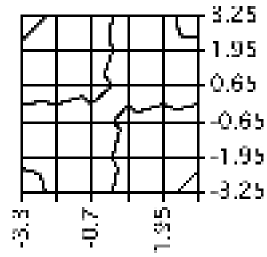
Member Stress 'σy' at Z=15m



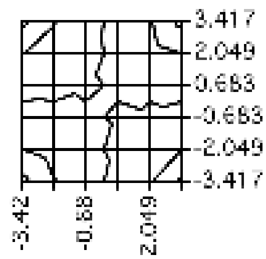
Member Stress 'σy' at Z=18m



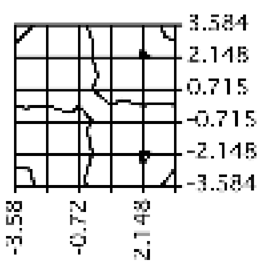
Member Stress 'τxy' at Z=12m



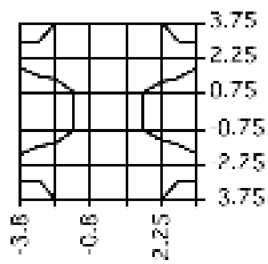
Member Stress 'τxy' at Z=15m



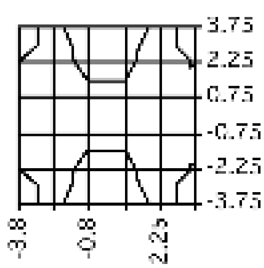
Member Stress 'τxy' at Z=18m



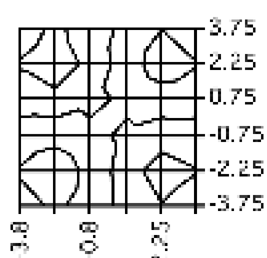
Member Stress 'σx' at Z=21m



Member Stress 'σy' at Z=21m



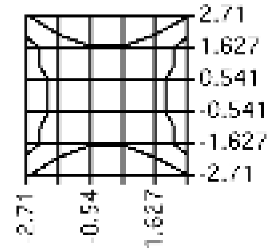
Member Stress 'τxy' at Z=21m



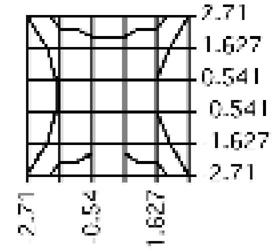
APPENDIX – IV

Flexural moments at various floor levels

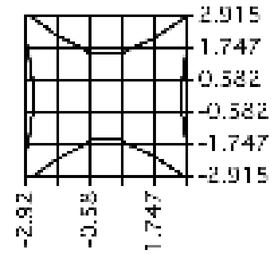
Member Stress 'σx' at Z=3m



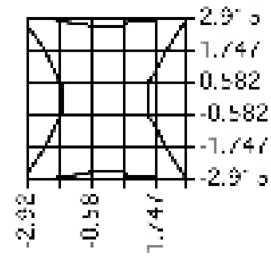
Member Stress 'σy' at Z=3m



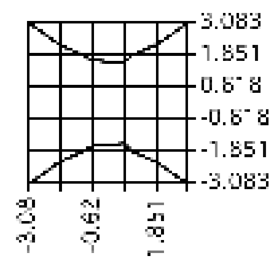
Member Stress 'σx' at Z=6m



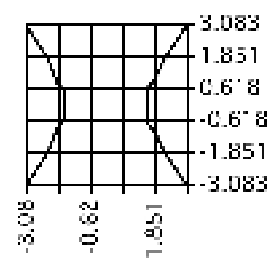
Member Stress 'σy' at Z=6m



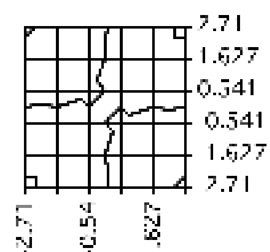
Member Stress 'σx' at Z=9m



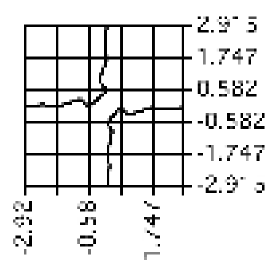
Member Stress 'σy' at Z=9m



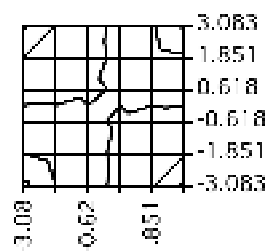
Member Stress 'τxy' at Z=3m



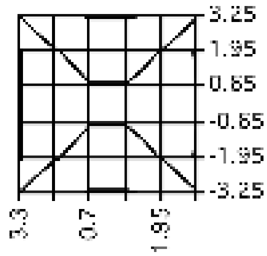
Member Stress 'τxy' at Z=6m



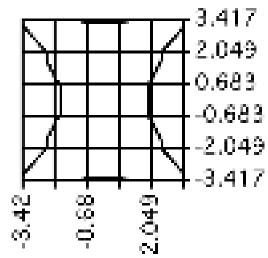
Member Stress 'τxy' at Z=9m



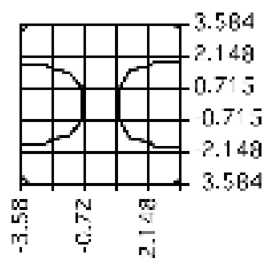
Member Stress 'σx' at Z=12m



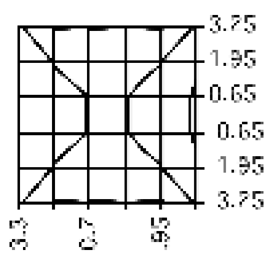
Member Stress 'σx' at Z=15m



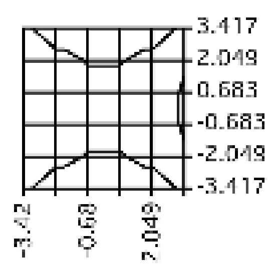
Member Stress 'σx' at Z=18m



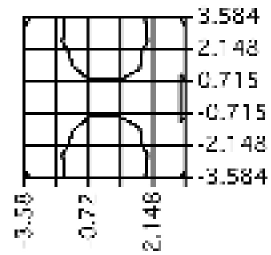
Member Stress 'σy' at Z=12m



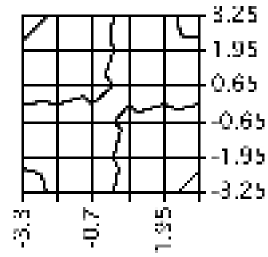
Member Stress 'σy' at Z=15m



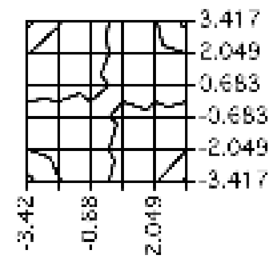
Member Stress 'σy' at Z=18m



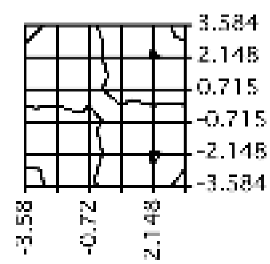
Member Stress 'τxy' at Z=12m



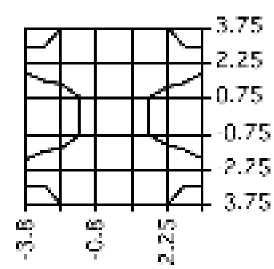
Member Stress 'τxy' at Z=15m



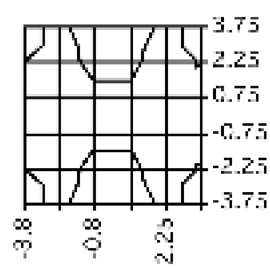
Member Stress 'τxy' at Z=18m



Member Stress 'σx' at Z=21m



Member Stress 'σy' at Z=21m



Member Stress 'τxy' at Z=21m