# STRUCTURAL ENGINEERING

QUARTERLY JOURNAL OF

INDIAN SOCIETY OF STRUCTURAL ENGINEERS

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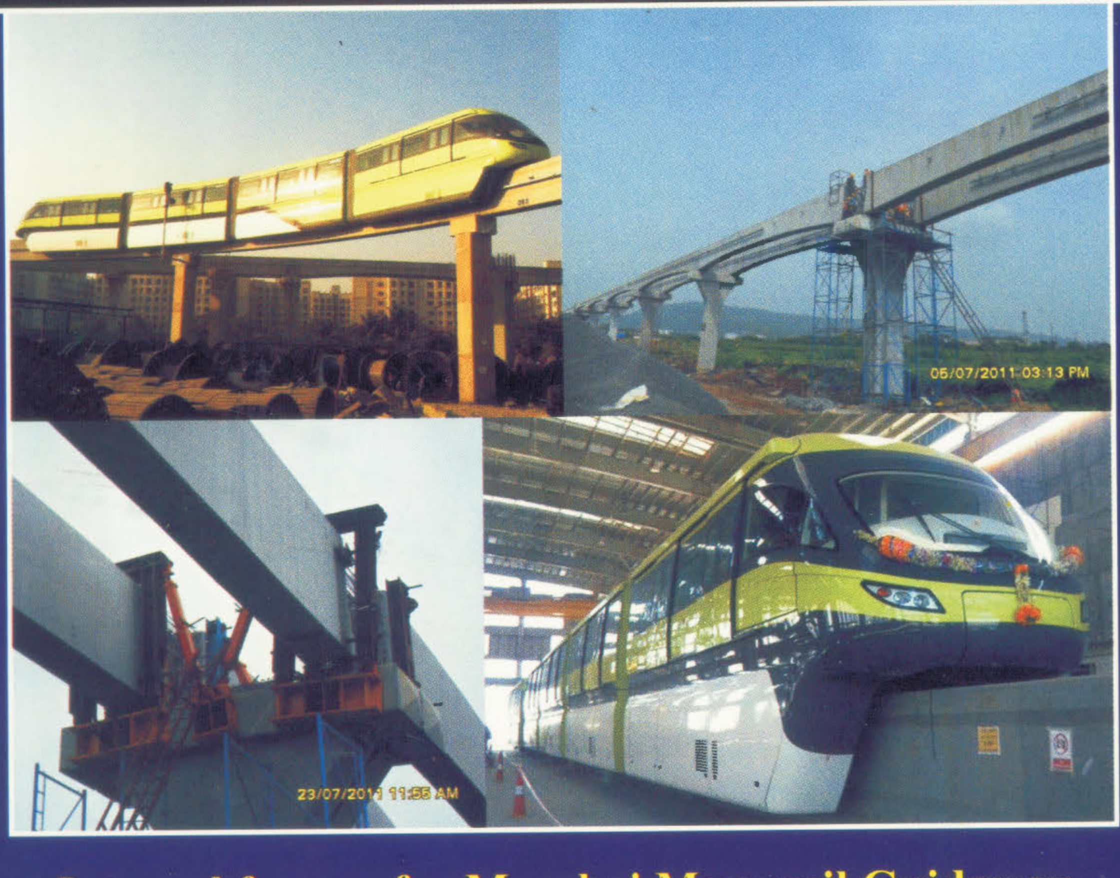
VOLUME 13-4

Oct-Nov-Dec-2011





**Integral frames for Mumbai Monorail Guideway**

(See Page 3 inside)

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**VOLUME 13-4, OCT-NOV-DEC 2011**

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1. To restore the desired status to the Structural Engineer in construction industry and to create awareness about the profession.
2. To define Boundaries of Responsibilities of Structural Engineer, commensurate with remuneration.
3. To get easy registration with Governments, Corporations and similar organizations all over India, for our members.
4. To reformulate Certification policies adopted by various authorities, to remove anomalies.
5. To convince all Govt. & Semi Govt. bodies for directly engaging Structural Engineer for his services.
6. To disseminate information in various fields of Structural Engineering, to all members.

# INTEGRAL FRAMES FOR MUMBAI MONORAIL GUIDEWAY

**Milind Bhoot, Elizer Abrea**

**Introduction:** Mumbai Monorail will to be the longest in the World after Chongqing and Disney. The Guideway Structures in the three projects belong to three different generations – integral frames, simply supported span concrete and steel beams respectively. Any structural engineer may appreciate that Mumbai project adapted the best of the prevailing practices in integral frames and then refined it.

Mumbai adapted the most accepted straddle-beam type Monorail system in which vertical and horizontal tires of the car straddle (hold) a single concrete beam from three faces against overturning. The guideway beam forms the track for the monorail cars with exposed concrete surface pre-cast with perfection of complex curve-surface geometry.

It was the pioneering advancement of construction techniques in Kuala Lumpur project that made the integral frames possible for Monorail. [3,4]Unique design and construction operations perfectly positioned the beams on curve geometry of the guide-way. However methods involved temporary clamping of frames at beam ends controlled by vertical and inclined hydraulic jacks from the pier-head which causes secondary prestress and corresponding time-dependent effects. It is essential to account for the temporary clamping effects in the model of structure on bridge-special software true to the construction arrangements.

Outline detail of Monorail Guideway Structures are published elsewhere[1,2] and this paper extends the sequel, fulfilling structural engineer's appetite, elaborating vital features of monorail guideway frame, distinguished from conventional bridge-structures. We recommend some background reading on Monorail; mostly available online (for better understanding of structural design concepts presented here.

We count on structural engineer's tendency to understand the new structure with reference to his previous expertise of the bridge structure. With such understanding, this paper compares Monorail guideway structures with bridges highlighting the distinguishing features.

**Benefits of Integral Frames** are well known to a bridge engineer. The integral frames were adapted in Monorail Guideway for substantial value addition with similar reasons. The savings on reduced number of bearings and expansion joints in monorail frames yield bigger dividends than the viaduct bridges, simply as monorail guideway beams need massive bearings and expansion joints. While the torsional restraint bearings are completely eliminated from the frame. The cantilevered steel fingers at expansion gaps are reduced drastically in numbers to above 25%

**Table : 1 Loads**

| **Loads** | **System generated Design forces** (From Monorail car Manufacturer) |
|---|---|
| Axle loads | 115kN, alternatively spaced @ 3.75m & 7.5m average. |
| Impact factor | Concrete beam: I = 20/(50+L) <= 25%, where L =span |
| Braking & Traction | 18% & 20%. |
| Hunting force | 25% |

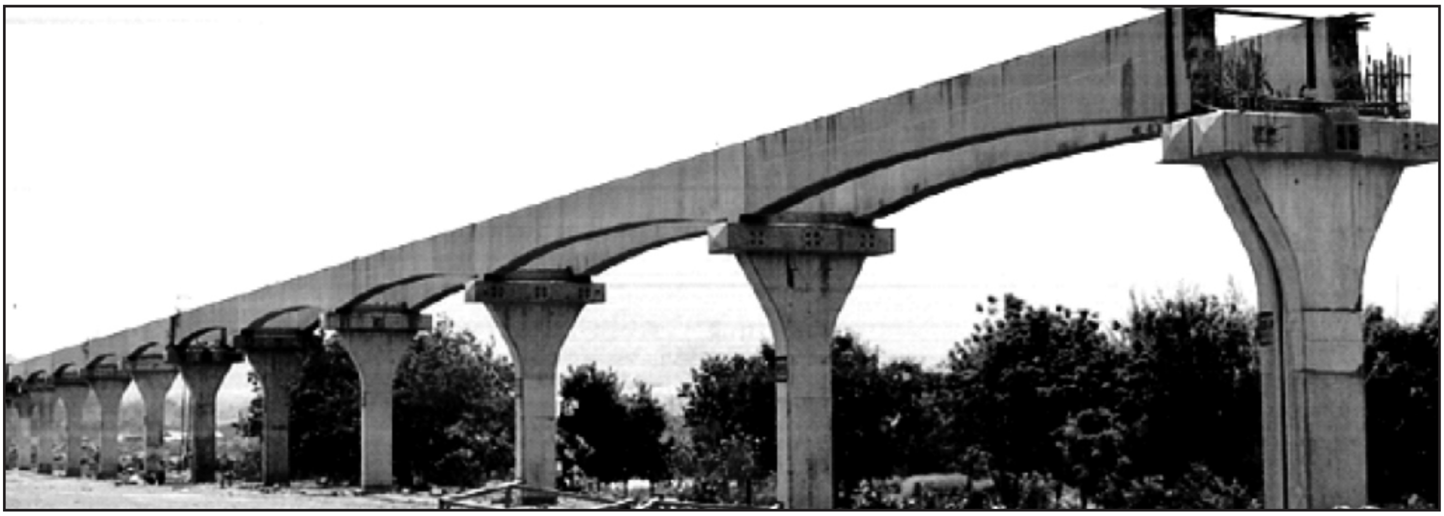

**Fig. 1 : Construction of Mumbai Monorail**

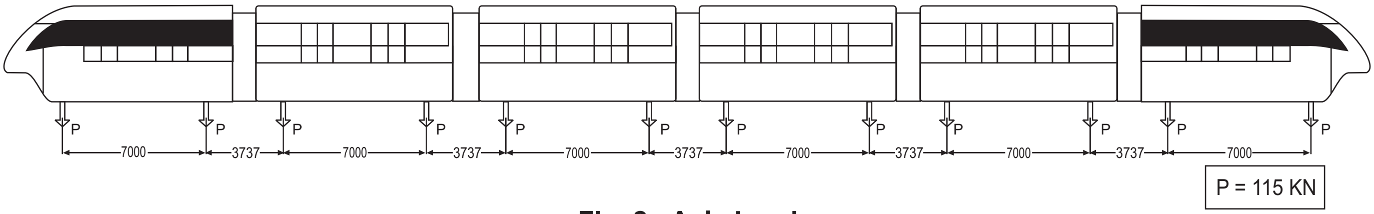


**Fig. 2 : Axle Load**

**Seismic forces**: Durability specifications of Mumbai Monorail for 120 years called for specific mix design requirements. In addition, up gradation of seismic design criteria from Zone 3 to Zone 4 accounting for higher probability of seismic occurrence over extended life-span and ensuing special rebar details ductility was necessary.

The structural engineer must correctly assess the lateral forces and overturning actions on the guideway beam caused in straddle monorail system.

The picture (Fig. : 3) illustrates how the lateral loads,- hunting loads, centrifugal forces etc should act on the Monorail Guideway Beams, a little differently than the normal steel-track rail cars. Similarly, the car-manufacturer may explain eccentricity of vertical loads from mechanical and construction tolerances. Half car loaded on one side of the beam center may be a critical condition in seismic and wind load combinations.

**Components of Integral Frames for Monorail Guideway:**

Monorail integral frame comprises of precast guide-beams of 23 to 27m spans to be 'stitched' in-situ with Y-piers of 12 to 20m height. The frame is finally post-tensioned by long continuity tendons integrating 3 to 5 spans.

Similarly, moment distribution in monorail frame gains bigger benefits than bridges, as the relative columns-beams flexibility permits more equitable distribution of forces in the frames. (See the comparison table 2)

**Pile foundations:** The typical two-pile arrangement compliments the integral frame arrangement very well. Moment-distribution becomes equitable and the added flexibility enhances the seismic response.

**Seismic shear stirrups in pile-caps**: The seismic detailing stipulations of IRS code were followed in the detailing with reference to zone-4.

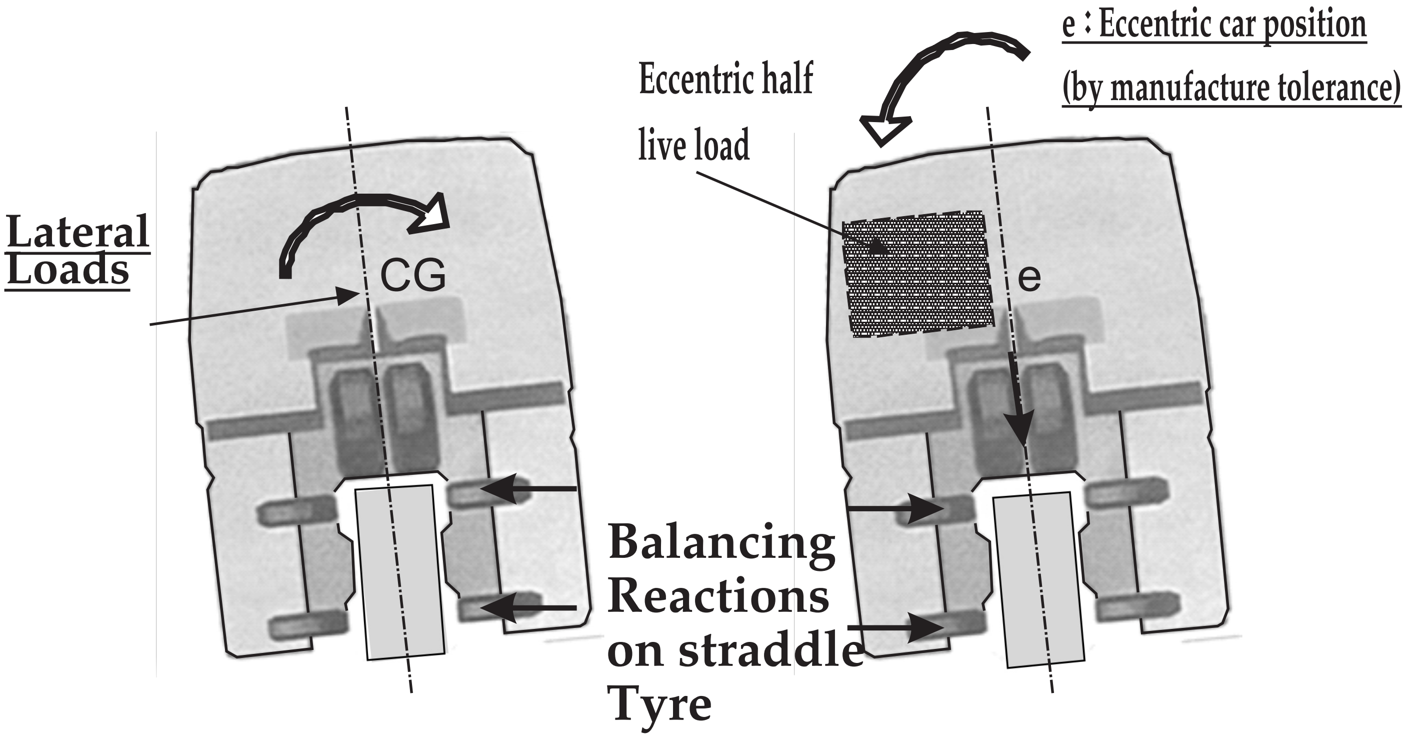


**Fig. 3 : Basic Load Actions on Guideway Beam**

However, the seismic detailing stipulations for pier-to-foundation connection are yet to be defined expressly in the present codes (though implied). This called for vertical seismic shear links in the body of pile-cap around the pier cage. Though already implied in the present IS code, Railway's seismic design guideline [5] now incorporates this with full expression.

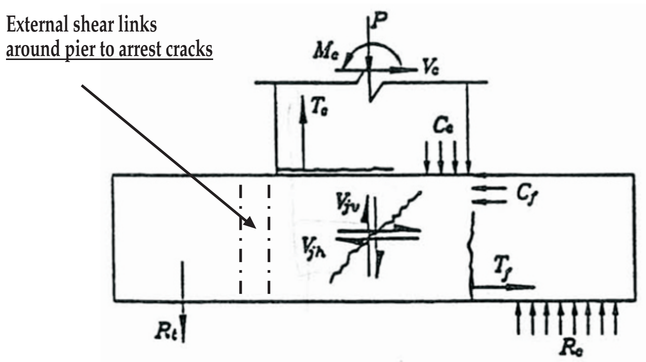


**Fig. 4 : Position of Stitches**

**Stitch member** (Frame joint) is a vital element integrating the monolithic frame. The stitch member is a 2.5m tall cast-in-situ stub projecting out of piers head. The Stitch serves as a vital monolithic joint between the precast beams with column. Designer face a severe limitation here in the design of the stitch as the contact area available between the pier-cap and columns is limited to just a RC section of 800x1000mm, too short to accommodate high concentration of complex force combinations, lateral (cantilever) bending, out-of-plane bending and torsion (i.e. forces in all 6 degrees of freedom).

Structural engineers often get confronted with highly congested reinforcements in such joints, to detail for special seismic ductility requirements - with a meticulous rebar clash analysis, meshing beam dowels into the confined core concrete of the stitch members.

The stitch interface with the precast beam is designed for shear-friction. Thanks to the prestress compression which adds to the shear-friction capacity (well only at the intermediate stitches, not at end stitches).

**End Stitches:** Compare the position of end stitches with the intermediate stitches in the in the Fig. 4. Intermediate stitches are sandwiched between the two span while the end stitches are left alone in a corner. As such, dead-loads from two precast guideway beams get balanced on the intermediate stitch joints while the end stitches remain unbalanced on dead loads of the end-span beams.

In addition, the intermediate stitches are boosted by the compression of continuity of pre-stress for high resilience against concentrated forces, while the end stitches remain un-tensioned RCC members. (The constructability constraints of continuity cables are terminated at the interface without intercepting the end stitches).

The end stitches are further constrained by reduced sections in accommodating expansion joint fixtures. While the intermediate stitches enjoy the full 800mm wide beam section, the end stitches are left with only 500mm, in the top 1200 depth; under the finger joints.

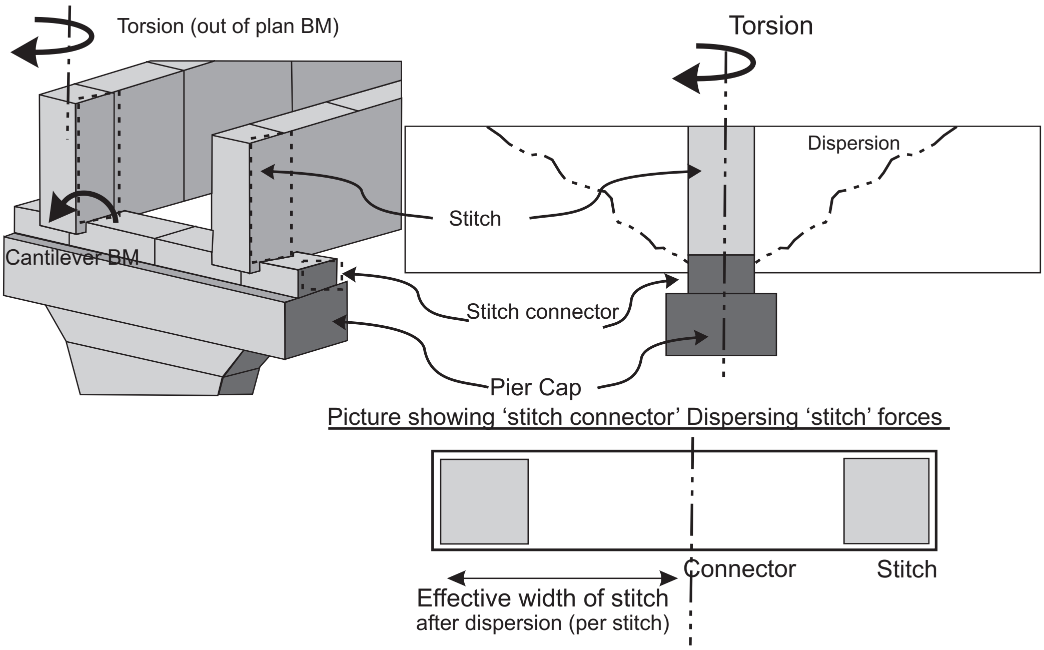


**Fig. 5 : Stitch Connector**

**Stitch connector:** As shown in Fig. 5, the pair of vertical stitch members is tied by a horizontal connector 450x1000 projecting monolithically over the pier-cap. The stitch-connector is a part of the pier-cap cast in second stage simultaneously with the vertical stitches. The precast guideway beams are embedded 250mm in such a way that the beam-column force transfer are not concentrated on the narrow 800x1000 stitch members and are effectively funneled (diffused) into a bigger section 800x2200 contact area between the pier and connector. To ensure this dispersion, it is necessary to embed the beam-dowels into the stitch-core confined with the main stitch bars.

With some deliberations, engineers may grasp the rationale of dispersion, an accepted resolution of a standard problem in Monorail integral frames.

**Diaphragms:** horizontal diaphragm slabs (3 numbers per span) substitute the conventional vertical diaphragms in the bridges. See Fig. 6 of computer model connecting pair of guideway beam bottoms.

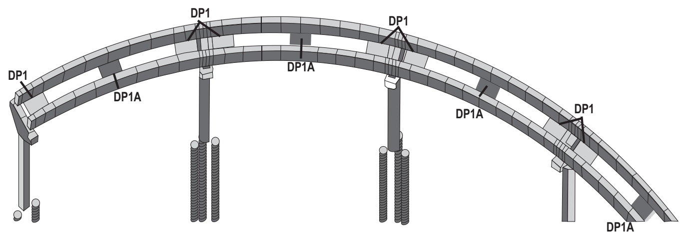


**Fig. 6 : Computer model of connecting pair of guideway beam bottom**

The 300mm thick diaphragm-slabs of 3mx3m size integrate the two beams against the lateral forces and torsion. The diaphragm adds lateral rigidity for enhanced performance of guideway beams reducing lateral deflections.

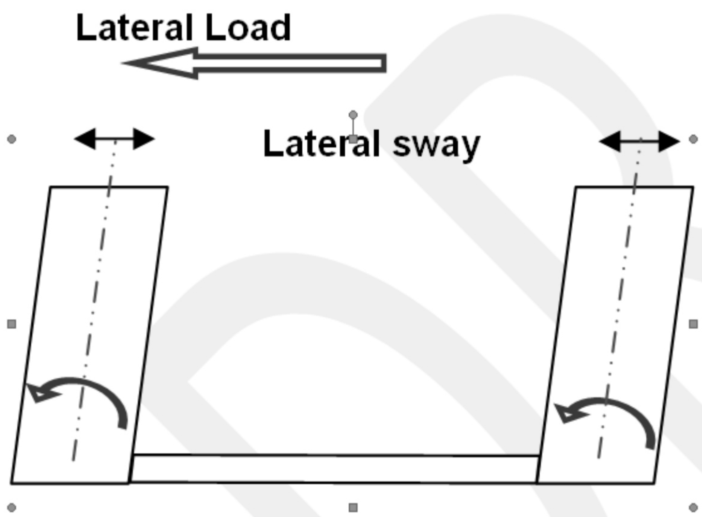


**Fig. 7 : U-section composite effect of 2 beams generated by diaphragm**

Diaphragm generates U-section composite effect of two beams for:

Restrained beam rotation and sway.

Resilience to loads (vertical / horizontal)

**Elimination of Diaphragms**: Mumbai Project eliminated all the diaphragms from the integral frame. The rationale is same as elimination of diaphragms from the modern bridge structures aimed at elimination of a tedious construction stage.

Lateral beam - sways were checked without the diaphragms on various curves by rationalizing the lateral forces as per the design speed-limits on the particular curve radii instead of using standard straight line design-speed on all curves. After such verification of the sway, to be within the permissible limit of 3mm the designers could eliminate the diaphragms.

The increased beam forces after removing diaphragms mean extra beam-rebar. This increased cost offsets saving in diaphragm-costs. Yet the time and trouble eliminated in the construction stages are appreciable, add a bonus - freedom from maintenance worries in Mumbai accumulating pigeon-waste over the horizontal diaphragm slab.

**Stages of construction :**

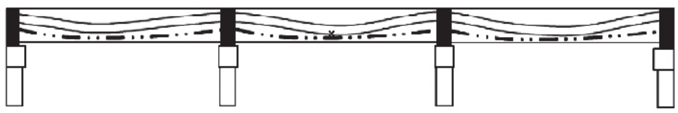

**Fig. 8 : Span & continuity post-tensioning arrangement**

Stage-1: Guide-Beams in **Precast**-Yard: Stress first two cables (bottom-dashed parabola marked in Fig. : 8) at 3$^{rd}$ day of precasting the beam in M60 grade concrete.
Stage-2: Beam **Launching and alignment**, after 14$^{th}$ days of pier casting in M40 concrete.
Stage-3: ***Intermediate stitch*** concrete M60 – (gap between beams shaded in Fig. : 8).
Stage-4: Stress the ***four continuity cables*** (frame length) after 10$^{th}$ day of Intermediate Stitches (continuous parabolic lines marked in the Fig. : 8).
Stage-5: Cast ***end-stitch*** in M60 concrete after inserting finger joint sole plates.
Stage-6: Construction of in-site diaphragms.
**Clamping stage** – Secondary effects**:**

Readers may re-visit the stage 3&4 written above. The beams are clamped temporarily in lateral and vertical direction to the pier-heads, while jacking the prestress on continuity cables. *The clamp-supports primarily meant for alignment purpose are necessary till the 'stitch' becomes strong enough to bear the continuity forces with the help of prestress*).

Temporary lateral clamp-supports (while pre-stressing) cause secondary lateral effects on the continuous beams. The secondary effects are only partially restored on removal of the clamps. Thus the structural designer must incorporate these temporary construction stages into the frame analysis model. Special bridge-purpose software calculates all the time-dependent effects accounting all micro-stages including such temporary construction effects in the frame.

**Table 2 : Comparison of Integral frames.**

| Comparison of Integral Frames for | | |
|---|---|---|
| | Typical Viaduct in Mumbai | Mumbai Monorail |
| 1. Deck | Stiff Deck 10x2m<br>(Box or Tee beams) | Flexible beam<br>0.8x2.2m voided |
| 2. Column | Column Height : 6m typical<br>Column size : 1x1.5m typical<br>: Typical 4-pile foundation. | Column Height : 10m typical (max 17m)<br>Column size : 1x1.5m typical.<br>: Typical 2-pile foundation.<br>Two pile arrangement further increases slenderness of piers improving the seismic response. |
| 3. Frame action | The deck rigidity compared to column is so high that the column-head is "fully fixed" against long forces.<br>And column contribution in the moment distribution at the deck joint is negligible. | The beam stiffness is comparable to column. Hence column head may rotate at the beam joint in proportion to its stiffness and attracts fair contribution in the moment distribution. |
| 4. Seismic Response | The small height and fixed head conditions make the column stiff, attracting higher seismic forces to the tune of 10% as calculated with IRC stipulations. | The slender flexible head column attracts seismic forces only say 4% as calculated with IRS-BR stipulations (IS 1893: 1994). |
| 5. Diaphragms | The vertical diaphragms are meant for improved grid action between the long beams. | Only horizontal diaphragms are permissible avoiding the path of straddle tires. The diaphragms generate U-section effect mainly adding lateral stiffness. |

The structural design sequence must permit release of clamps from the intermediate supports; before completing the end stitches. The clamps on end stitches may be released only after installing sole-plates, (fixture plates underneath expansion joint fingers). It is reasonable to release the intermediate clamps for reuse at other frames for optimum use of construction-resources.

The table 2 explains why the monorail structure is flexible in comparison to the normal viaduct bridges. Hence attracts smaller seismic forces such that wind force generally governs over seismic forces (though the seismic detailing is mandatory)

**Frame-analysis:** Monorail structures are fine examples of how integral frames are designed and skillfully optimised on the modern bridge design software. Such software allows auto generated moving train, post-tensioning forces, losses and tendon extensions etc. As explained above, the time dependent analysis for the creep and shrinkage effects accounts for construction stages of pre-stressing phases and changes in support conditions during handling stages.

Analysing prestressed integral frames is no more a lengthy cumbersome exercise. The modern bridge software takes care of the construction stage analysis, phasing out all the time dependent effects like creep and shrinkage.

**Construction sequence drawings:** The last paragraph explains very well, why construction method statements are completed in consultation with the structural engineer before the designs are finalized. Construction sequence drawings define the stages of constructions and corresponding assumptions of concrete stages. The structural engineer accounts for the pre-determined construction sequence for phasing out the time dependent effects in the same sequence. Once defined on the drawings, the sequence must be adhered at site; acknowledging the design implications. Any temporary clamping of elements involved in the enabling structures must reflect into construction sequence drawings. In short, the construction sequence drawings are to be taken as part of the structural design for integral frames, as changes in construction stages means revision of design assumptions.

**Portal Frames with sliding bearing supports**: Portal beams support the guideway beam with unidirectional

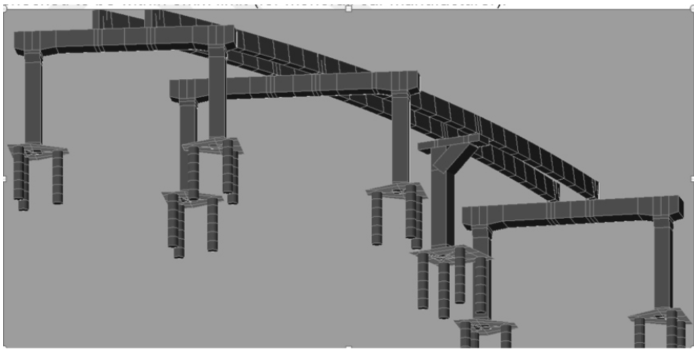

**Fig. 9 : Model in Member Bridge Design Softwars for design of Integral Frame.**

sliding bearings (Pot-PTFE), releasing moments in all 3 directions. Releasing the guideway beam supports of long moments means release of torsion reactions on the portal beams, in perpendicular direction.

As one of the beam support within the frames is released for lateral rotation, effective unsupported lateral span length of beam-top increases to two span lengths. For the given stiffness of beams, the increased lateral deflection of beam-top was cross-checked to be within 3mm limit (for monorail car manufacturer).

**Portal Frames with torsion restraint bearings**: In continuity with explanations given above, releasing transverse rotation of two consecutive guideway beam supports would increase the lateral unsupported span to 3 span lengths and lateral sway of the beam top will exceed the permissible limits stipulated by monorail cars manufacturer. (Which means the lateral sway of the car will exceed the permissible limits). In addition, there will be excessive torsion in the guideway beams to accommodate in the structural designs.

The Mumbai Monorail crossing of Eastern-Freeway required 8 consecutive portal frames. Providing moment release bearings at all 8 portal supports meant unsupported lateral span longer than 200m. Thus torsion restraint bearings were required to limit the lateral unsupported spans within 55m.

As mentioned earlier, the torsion restraint bearing used historically in monorail simple spans were provided in Mumbai project for only limited locations. The torsion restrained bearings, designed as per BS5400 in Monorail, are comparable to roller steel bearings, well known in Indian Railways for old steel bridges. As we know, these are massive bearings 1200x800x500m weighing 3 tons. Long torsion restraint anchor-bolts clash with congested stitch rebar adding to the problems of handling at the time of construction.

**Conclusions**: After the experience of Monorail, Indian engineers are likely to design more integral bridges for rails and roads. The impact of such value engineering has far reaching implications on the way we design and build our structures. Modern tools used in Monorail have potential to develop the cutting edge capabilities, competitive with the global standards.

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**Reference :-**

1. Elizer Abrea and Milind Bhoot, on “Mumbai Monorail – Blending of systems with Guideway Beams,” Konstruction Review” Sep 19, 2011 (digital
2. C. Shankarlingam, “Construction Methodologies for Mumbai Monorail Project, “Civil Engineering & Construction Review, April 2011
3. Vishnu Kumar and Milind Bhoot, “Design and Construction of Monorail Guideway Beams,” IIBE Symposium, Hyderabad July 2011
4. Roland von Wölfel & Manfred Braun- The Guideway System of Monorail Kuala Lumpur– Design & Erection, IABSE Symposia, Shanghai 04
5. IITK-RDSO Guidelines for Seismic Designs of Railway Bridges. Provisions with commentary and explanatory Examples. October 2010.

# POST TENSIONING OF CABLES IN CABLE STAYED BRIDGES

**Ravindra.R* and Prof.V.Devaraj****

*Estimation of post tensioning forces in cable stayed bridges forms an important phase in the analysis of cable-stayed bridges. Post tensioning forces are evaluated by displacement reduction method. The cable attachment points to the deck and the topmost cable attachment points to the pylon are considered as control points. The post tensioning forces are so evaluated that the displacement at the control points can be controlled under dead load condition.*

## 1. Introduction:

Cables form an important component of cable-stayed bridges. The deck of the bridge is supported directly from the pylon with inclined cables, which are always in a state of tension. Cables used as stays in cable-stayed bridges are generally available in five types – parallel solid bars, parallel wires, parallel strands, locked coil strands and ropes shown in Fig.1.

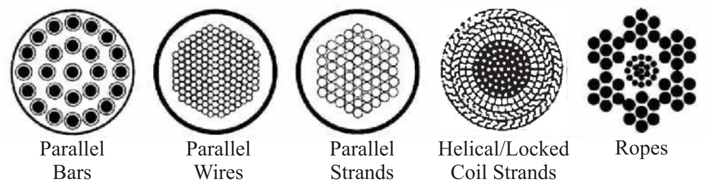


**Fig. 1 : Types of cables used as stays**

The strands are pre stretched to remove the looseness (constructional stretch). The twisted strands get adjusted into a denser cross section. Pre stretching enables the designer to predict the elastic behavior of the cable after erection in structure.

In design offices, the post tensioning forces are normally found by trial and error basis, which involves much iteration, thereby consuming a lot of designers time. The post tensioning forces can be found by a simpler and a straightforward method called the 'displacement reduction method' using stiffness matrix method. Post tensioning of cables has the following advantages:

a) It increases the axial stiffness of the cable as the sag gets reduced.

b) It reduces the deflection at the cable attachment points making the deck to behave like a continuous beam elastically supported by cables.

c) The structure becomes stiffer.

## 2. Non-linear behavior of cable stayed bridges:

A cable-stayed bridge is a structural system in which the deck is elastically supported at points along its length. One of the difficulties, which an engineer encounters while designing a cable-stayed bridge, is its behaviour, particularly non-linear behaviour under normal loads. It is highly desirable in bridge engineering to develop accurate procedures that can lead to a thorough understanding of static and dynamic behaviour. Even though the material in the members of a cable stayed bridge behaves in a linear elastic manner, the overall load-displacement relationships for the structure will be nonlinear under normal design loads. The source of non-linearity originates from: i) Non linear behaviour of the cables due to the sag in the cables. ii) Nonlinear behaviour of the bending members: i.e. the deck and pylon due to the coupling of axial compression and bending moment. iii) geometry change in due to large displacement.

## 3. Non-linear behavior of cables:

When a cable is suspended from its ends and subjected to its own weight and an externally applied axial tensile force, it sags into a shape of catenary. The displacements of the cable ends, which result from the deformations in the structure due to the applied loads, have three distinct effects. The first is the change in strain in the cable material. This change in strain can be considered to be linear and is governed by modulus of elasticity. Second there is a rearrangement of the individual wires in the cable cross section under changing load. Part of this deformation, known as constructional stretch is permanent deformation usually eliminated by the cable manufacturer during the manufacturing process, by pre stretching the cable to a load higher than working loads. The non-permanent part of this deformation results in an apparent reduced effective modulus of elasticity of the cable material. Thirdly there is a change in sag of the cable, exclusive of material deformation. The length of the cable governs this change in sag. It is this change in sag, which varies nonlinearly with cable tension, causes non-linear force deformation relationship. The axial stiffness of the cable varies nonlinearly as function of end displacements. Part of the end movements occurs due to material deformation and another part occurs due to change in sag. The change in sag becomes smaller as the axial tension increases and the end movement occurs due to material deformation. The axial stiffness of the cable increases as its tensile stresses increases. A convenient method for considering the non-linearity in the inclined cable stays is to consider an equivalent straight chord member with an equivalent modulus of elasticity. The equivalent modulus of elasticity combines both the effect of material and geometric deformations. This concept was first introduced by Ernst and is well verified by many investigators [John Fleming 1979, Aly and Ahmed 1990]. The equivalent modulus of elasticity is given by

$$E_{eq} \quad \frac{E}{1 \quad \frac{W L^2 A E}{12 T^3}} \qquad \text{... (1)}$$

E = effective modulus of elasticity, L= horizontal projected length of the cable, W= weight per unit length of the cable, A = cross sectional area of the cable, and T = cable tension.

The element stiffness matrix for a cable element in local coordinates by using the concept of equivalent modulus of elasticity is

$$K_e \quad \frac{AE_{eq}}{L_C} \begin{matrix} 1 & 1 \\ 1 & 1 \end{matrix} \qquad \ldots(2)$$

Where $L_C$ = chord length of the cable

**4. Displacement reduction method:**

The essential principle behind this method [Benjamin E Lazar, et.al. 1972, M.S.Troitsky and Lazar 1971] is to obtain an appropriate scheme of tensile cable forces. The resulting bending moments, in the deck then, are essentially those of an equivalent continuous beam with all supports from cables and towers considered as rigid simple supports. The procedure to calculate the post tensioning forces involves first determining the matrix of displacements {DJ} at a number of n selected locations, where n is the number of cables. The displacements are due to a unit force applied successively along each cable of the bridge system. The size of this matrix will be (n x n). Further, the vector {J} of displacements due to dead load at the locations selected is determined. If $C_o$ is the reduction factor, then

$\{J\}+ [DJ].\{X\}= C_0.\{J\}$ …(3)

where {X}is the vector of post tensioning forces in cables. From the above equation {X}may be determined as

$\{X\}= (C_o – 1). [DJ]^{-1}.\{J\}$ …(4)

Post tensioning forces to be found by displacement reduction method [Benjamin E Lazar, et.al. 1972, M.S.Troitsky and Lazar 1971] requires an analysis program of the cable stayed bridge to be developed. The authors have developed a program in MATLAB, to compute the post tensioning forces and the cables forces considering the non-linearity in the cables, deck and pylon. The model of the bridge undertaken for analysis is shown in Fig. 2 .The dead load considered are the self weight and a superimposed dead load of 53.66kN/m. The flow chart of the program is shown in Fig.3.The deflections at the control points due to dead load can be obtained from the program. The control points considered in the analysis are the cable attachment point of the deck (node 1 to 18) and the backstay cable attachment point to the pylon (node 23). The reduction factor, $C_0$ is less than one**.** If no displacement is desired then the value of $C_0$ is set to zero, which is considered here. The vertical displacement at the control points of the deck and the horizontal displacement at the control point of the pylon are shown in Table 1. The equivalent modulus of elasticity $E_{eq}$ depends upon cable tension. The initial values of cable tension is obtained from the first cycle of analysis considering the cable as axially loaded elements and the corresponding stiffness matrices without incorporating equivalent modulus of elasticity, $E_{eq}$. The cable tension so obtained is used to compute equivalent modulus of elasticity $E_{eq}$ (shown in Equation.1) in the second cycle of analysis to

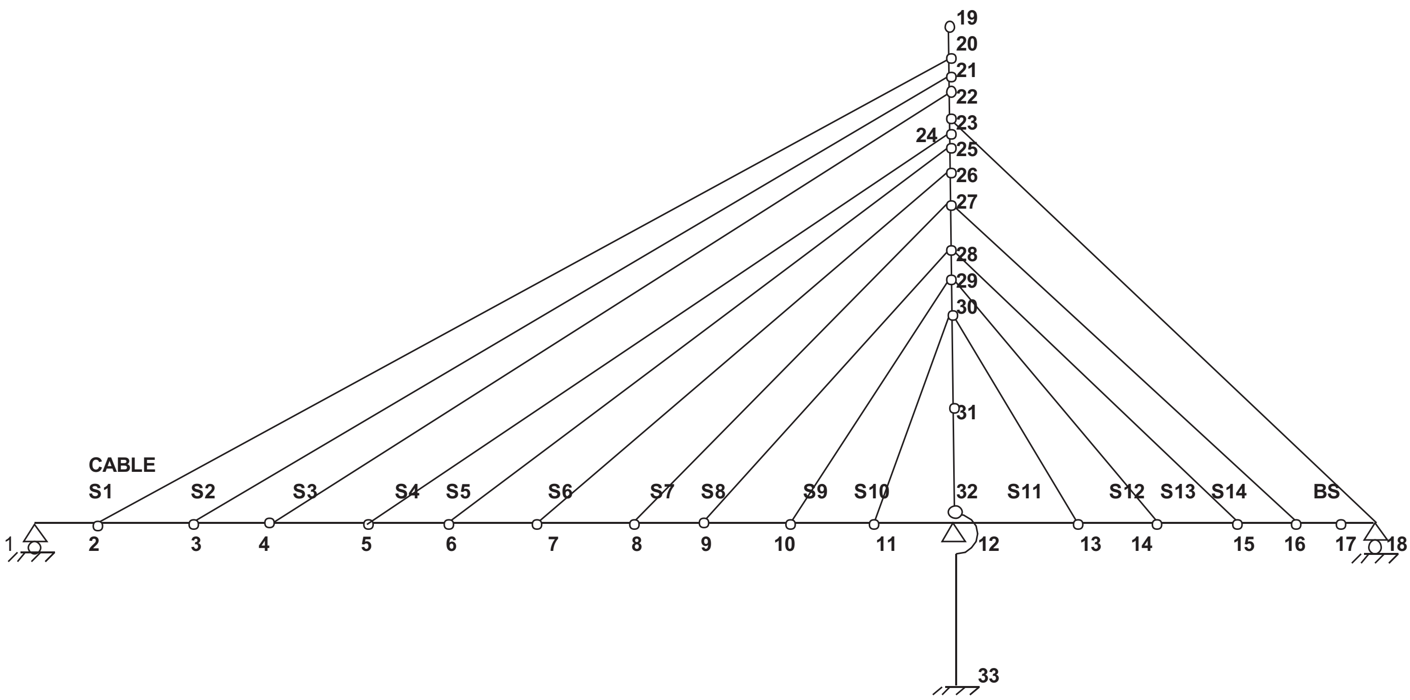


**Fig. 2 : Modal of the bridge**

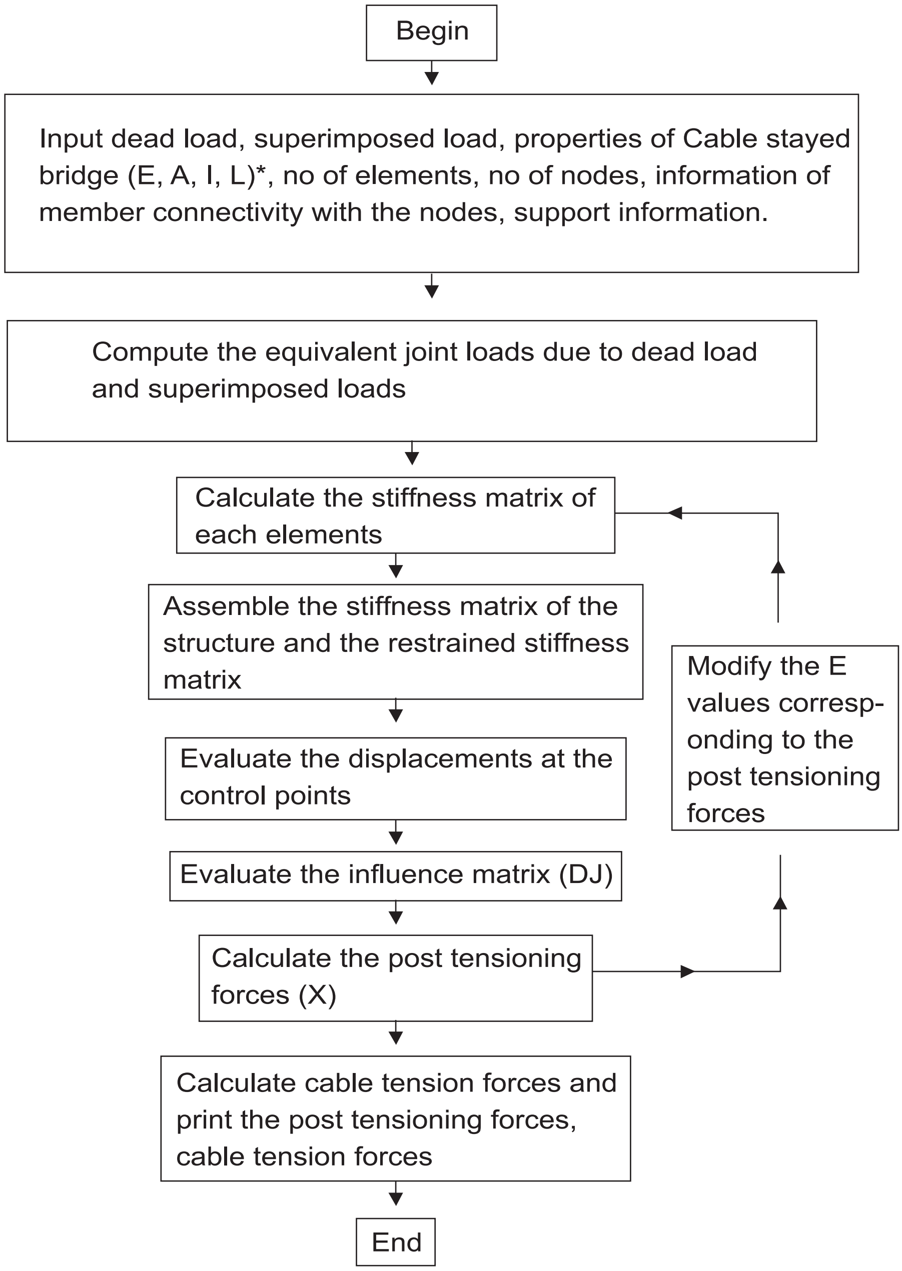


**Fig 3 : Flow chart of the program to evaluate post tensioning forces and cable forces**

(*E=modulus of elasticity, A= area of cross section, I=Moment of inertia, L=length of member)

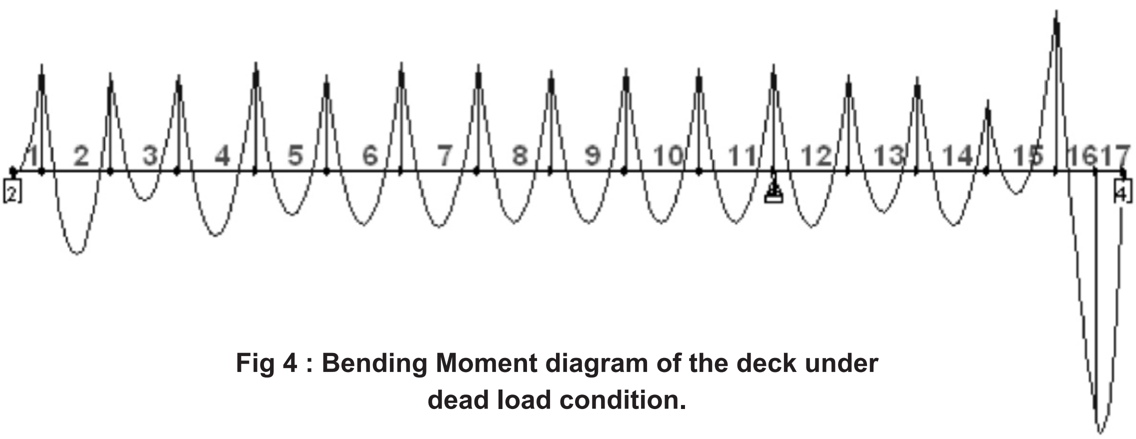


**Fig 4 : Bending Moment diagram of the deck under dead load condition.**

give the cable tensions for the desired displacement condition. The cable forces and the post tensioning forces obtained from analysis using MATLAB is shown in Table 2.The cable stayed bridge is also analysed by StaadPro analysis software considering the values of post tensioning forces obtained from second analysis cycle. The cable forces from analysis using MATLAB and StaadPro agree well as seen in Table 3.The variation in the values of equivalent modulus of elasticity, $E_{eq}$ of the cables is also shown in Table 3. The cables act as a support to the deck and the deck behaves like a continuous beam, which is evident from the bending moment diagram shown in Fig.4

## 5. Conclusions:

The displacement reduction method is a simple straight forward method to compute the post tensioning forces in cables of cable-stayed bridges, so that the deck behaves as a continuous beam with desired displacement constraints at cable attachment points. The set of post tensioning forces obtained by displacement reduction method can be taken as initial tensions for cable elements, for analysis of cable stayed bridges using Staad Pro analysis software.

## 6. References:


1) Aly S Nazmy and Ahmed M Abdel-Ghaffer, 'Three dimensional Non Linear Static Analysis Of Cable-Stayed Bridges', *Computers and Structures*, Vol 34, No 2, 1990, page 257-271.
2) Benjamin E Lazar, Michael S. Troitsky and Matthew McC.Douglass, 'Load balancing analysis of cable stayed bridges', *Journal of Structural Division, Proceedings of American Society of Civil Engineers*, Vol 98, No ST8, Proc. Paper 9122 August 1972,page 1725-1740.
3) John F Fleming,' Non Linear Static Analysis Of Cable-Stayed Bridge Structures', *Computers and Structures*, Vol 10 1979, page 621-635.
4) M.S.Troitsky and B.E.Lazar, 'Model analysis and design of cable stayed bridges', Proceedings, *The Institution of Civil Engineers,* March 1971, page 439-464.


**Table 1 Displacements at nodes under dead load condition**

| Nodes | dx | dy | teta |
|---|---|---|---|
| 1 | 0.009811 | 0 | 2.39E-06 |
| 2 | 0.009811 | 3.15E-09 | -8.47E-06 |
| 3 | 0.00964 | 1.01E-08 | 9.59E-06 |
| 4 | 0.009184 | 1.45E-08 | -9.38E-06 |
| 5 | 0.008462 | 1.55E-08 | 5.98E-06 |
| 6 | 0.007615 | 1.33E-08 | -2.74E-06 |
| 7 | 0.006592 | 8.98E-09 | -5.27E-07 |
| 8 | 0.00541 | 4.21E-09 | 1.26E-06 |
| 9 | 0.004152 | 5.55E-10 | -1.97E-08 |
| 10 | 0.002821 | -1.31E-09 | -3.60E-07 |
| 11 | 0.001426 | -1.33E-09 | 3.24E-07 |
| 12 | 0 | 0 | -1.26E-06 |
| 13 | -0.00144 | 1.78E-09 | 3.97E-06 |
| 14 | -0.00276 | 2.99E-09 | -3.14E-06 |
| 5 | -0.00403 | 3.17E-09 | 8.51E-06 |
| 16 | -0.00524 | 1.98E-09 | -3.09E-05 |
| 17 | -0.00586 | -0.00016 | 3.46E-05 |
| 18 | -0.00593 | 0 | 4.77E-05 |
| 19 | -0.00225 | -0.0086 | 0.000572 |
| 20 | -0.00116 | -0.0086 | 0.000572 |
| 21 | -0.00067 | -0.00859 | 0.000564 |
| 22 | -0.00044 | -0.00858 | 0.000552 |
| 23 | -1.53E-08 | -0.00856 | 0.000502 |
| 24 | 0.000231 | -0.00851 | 0.000466 |
| 25 | 0.000628 | -0.00841 | 0.000415 |
| 26 | 0.00102 | -0.00829 | 0.000375 |
| 27 | 0.001364 | -0.00818 | 0.000341 |
| 28 | 0.001668 | -0.00804 | 0.00031 |
| 29 | 0.001859 | -0.00794 | 0.000289 |
| 30 | 0.001924 | -0.0079 | 0.000282 |
| 31 | 0.003309 | -0.00623 | 4.41E-05 |
| 32 | 0.00072 | -0.00161 | -0.00011 |
| 33 | 0 | 0 | 0 |

(dx:horizontal displacement in meters, dy:vertical displacement in meters, teta:rotation in radians)

**Table 2 Cable forces and post tensioning forces from analysis using MATLAB**

| Cable | Cable Forces* kN | | Post Tensioning Forces ,kN | |
|---|---|---|---|---|
| | First cycle | Second cycle | First cycle | Second cycle |
| S1 | -5504.90 | -5505.05 | 5749.53 | 5749.92 |
| S2 | -4897.10 | -4896.98 | 5124.65 | 5124.67 |
| S3 | -3989.70 | -3989.76 | 4210.15 | 4210.30 |
| S4 | -3876.29 | -3876.31 | 4070.67 | 4070.63 |
| S5 | -3333.05 | -3333.00 | 3539.56 | 3539.36 |
| S6 | -3166.27 | -3166.30 | 3359.50 | 3359.27 |
| S7 | -2772.33 | -2772.33 | 2959.66 | 2959.30 |
| S8 | -2395.01 | -2395.00 | 2563.52 | 2563.12 |
| S9 | -2136.17 | -2136.18 | 2303.56 | 2303.22 |
| S10 | -1939.76 | -1939.74 | 2081.79 | 2081.60 |
| S11 | -1876.75 | -1876.70 | 2037.89 | 2038.02 |
| S12 | -2018.32 | -2018.45 | 2196.23 | 2196.67 |
| S13 | -2003.81 | -2003.52 | 2209.03 | 2209.12 |
| S14 | -3773.94 | -3774.26 | 3929.57 | 3930.15 |
| BS | -28686.00 | -28687.64 | 29636.41 | 29638.08 |

**Table 3 Cable forces by MATLAB program and StaadPro**

| CABLE FORCES IN kN | | | | Equivalent modulus of elasticity of cables, Eeq (kN/m$^2$) |
|---|---|---|---|---|
| Cable | MATLAB | StaadPro | Percentage difference | |
| S1 | -5505.05 | -5505.55 | 0.009 | 198876392.72 |
| S2 | -4896.98 | -4897.24 | 0.005 | 199091741.01 |
| S3 | -3989.76 | -3989.97 | 0.005 | 198960048.06 |
| S4 | -3876.31 | -3876.39 | 0.002 | 199442018.98 |
| S5 | -3333.00 | -3333.02 | 0.000 | 199308043.97 |
| S6 | -3166.30 | -3166.27 | -0.001 | 199581106.60 |
| S7 | -2772.33 | -2772.25 | -0.003 | 199686627.84 |
| S8 | -2395.00 | -2394.90 | -0.004 | 199832157.99 |
| S9 | -2136.18 | -2136.09 | -0.004 | 199921308.52 |
| S10 | -1939.74 | -1939.70 | -0.002 | 199988186.88 |
| S11 | -1876.70 | -1876.75 | 0.003 | 199987300.68 |
| S12 | -2018.45 | -2018.56 | 0.006 | 199946300.75 |
| S13 | -2003.52 | -2003.66 | 0.007 | 199788543.01 |
| S14 | -3774.26 | -3774.36 | 0.003 | 199963844.97 |
| BS | -28687.64 | -28686.91 | -0.003 | 199949237.75 |

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# JACK-UP PLATFORM FOR MARINE EXPLORATIONS

**D. J. KETKAR**

The first platform was designed, built and used in the year 1967 at Madras (now Chennai) for sea-bed soil investigation in shallow waters. This was much before the arrival of 'Sagar Samart' of ONGC for deep water operation.

Conventionally, such soil investigation is carried out using floating craft. Usually 6 anchors are used for mooring the craft in position. These anchors need continuous manouvering of their ropes so that with rising or falling tides, the anchor-ropes remain tight. This avoids side-drifting of the craft.

Another tricky problem, faced at Madras, was that the anchors in the sandy sea-bed, used to loose their embedment when tide turned from high to low. During advance of low- tide, sea-bed sand used to flow away from shoreline and anchors used to get free. This resulted loss of holding down support of the craft.

To carry out work of soil investigation, it is necessary to position the drilling rig at the location of the borehole. When anchors get loosened, the craft shifts from its position and thereby the position of the drilling rig also get shifted.

Hence, it becomes necessary to use a craft with spuds or columns. Spuds keep the craft in proper location by controlling the lateral movement, but the vertical up and down tidal movement is not controlled. Jack-up legs act as columns and support full weight of the craft. Even with limited resources, it was decided to go for a jack-up platform and manufacture it at a local workshop, at Madras.

**Brief Details of the Jack-up**

a) Legs - 6 Nos 40' long (12 m) each made of 36" x 6" (90 x 15 cm Rolled steel I sections) with 3' x 3' ( 0.9 x 0.9 m ) base plates.

b) Platform - 18' x 36' (5.4 x 10.8 m) x 6' (1.8 m) deep.

- Supported on steel trusses to form boxes.

- For floating, 3' diam x 9' long cylinders x 10 nos. were positioned in truss boxes.

Pay load (floating)
Capacity = 30 T
Max. depth of water = 7 m

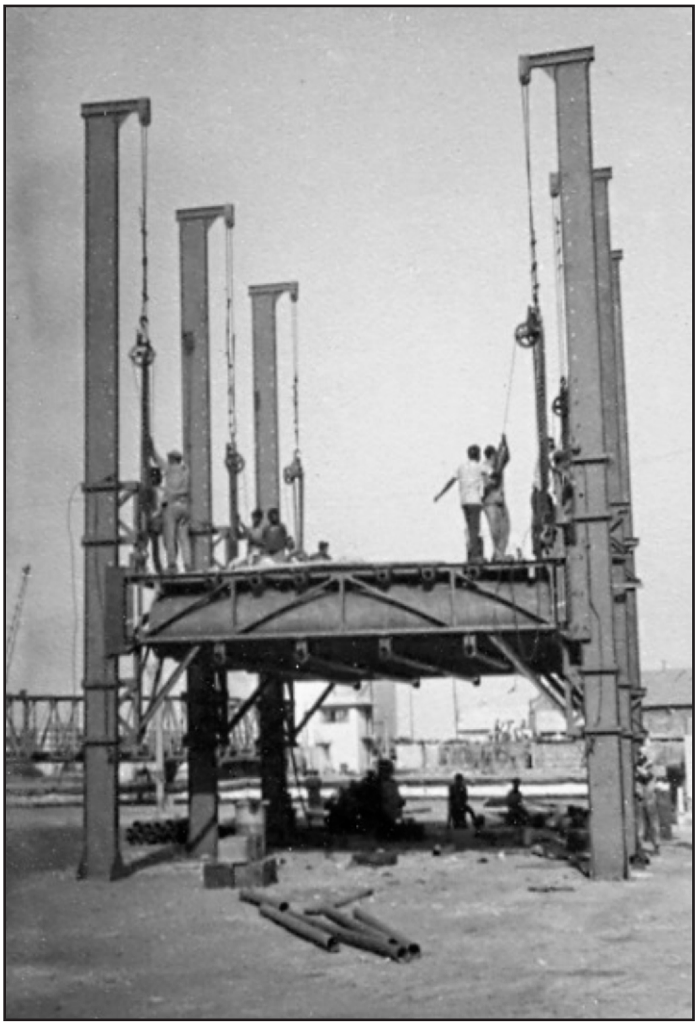

**Fig. 1 : Jack-up Platform assembled at the Jetty (For subsequent Lift-off and placing on water)**

c) Working Well – A central well of 3' x 7' (0.9 x 2.1 m) was provided in the platform

d) Leg guides – 6 nos. guides fitted with rollers were fixed along vertical Sides of the platform.

e) Operation of Legs – From 10' (3m) high columns erected on platform. A system of pulleys and wire ropes were installed on each leg. 6 nos. 2 T capacity winches were fixed on the platform near the base of each column to operate the ropes.

Performance: The platform was successfully used for drilling 9 holes in the seabed.

## 3) OPERATION OF THE JACK-UP

a) Lift the spuds, upto just underside of the platform and lock them in position.

b) Float the Jack-up and shift it to its position.

c) Lower down the spuds-to penetrate the seabed. Position all 6 anchors.

d) Using ropes on Pulleys (L) and (T) raise the platform about 1.0 m above the high tide level. Insert locking pins ( in all 6 spuds) to hold the platform in raised position.

e) For shifting the Jack-up to another location-remove the locking pins, lower the platform to rest on water, raise the spuds, using the locking pins hold the spuds in position . Then shift the Jack-up, using the anchor ropes.

### 3.1 Modules of the Platform

a) 12 M long spuds – 6 nos

b) 0.9 m dia m x 2.7 m long cylindrical floats – 10 nos.

c) 5.4 m long x 1.5 m high trusses - 12 nos.

d) Bollards for marine anchors – 6 nos.

e) Marine anchors on ropes – 6 nos.

f) 10 T capacity chain pulley blocks – 6 nos.

g) 0.9 m x 1.8 m deck plates – 36 nos.

h) Spud guide frames – 6 nos.

i) Spud shoes – 6 nos.

j) 2 T deck-winches – 6 nos.

### 3.2 Platform Assembly on a Warf

a) Assemble Trusses by bolting to form the platform box.

b) Place the Cylindrical floats, between the Trusses.

c) Provide top members to fix the Floats in place.

d) Bolt the spud guide frames to the side Trusses.

e) Place the Spud-shoes under the spud-guides.

f) Erect the spuds on the guide frames using guy ropes for temporary supports. Fix lifting ropes near the shoes.

g) Hook up the chain pulley blocks to top of spuds and the platform.

h) Raise the platform, by say 2 m above the base.

i) Fix Deck plates, Bollards, etc on the platform.

### 3.3 Launching in Sea

a) Using a floating crane, lift the platform from the wharf.

b) Transport the platform near its location and place it on water, still suspended form the floating crane.

c) Lower the spuds, using the deck-winches and pulleys (L) onto the seabed.

d) Using a powered boat, drop marine anchors on the seabed, but sufficiently away from the platform. Bring the anchor rope ends and fix them to the bollard drums. Tighten the ropes by winding them on the bollard drums and lock the drums.

e) Immediately raise the platform, using chain pulley blocks (T) to a level about 1.0 m above the high tide level.

Engage the locking pins ( on the platform) by inserting them in appropriate holes in the respective spud legs.

### 3.4 Positioning the platform at borehole location

a) Establish a base-line on the shore using flags.

b) Mark 2 points on the base-line, at measured distance from the existing survey-grid lines. Fix 30 cm high marker pegs, on these points, with black and white stripes.

c) Float the platform on water, but held by 6 rope anchors.

d) At the central well on the platform deck, place a painted pole near the centre of the well.

e) Place 2 Theodolities, one each above the markers pegs (fixed in 'b' above)

f) For each Theodolite position, calculate the horizontal angle, with respect to the base-line, to the borehole location.

g) Take sightings from each theodolite onto the ' painted pole', (placed as mentioned above). Using the rope anchors, shift the platform to correct borehole location.

h) Using chain pulley blocks/ (T) and spuds raise the platform about 1.5 m above the expected high tide level. Then lock it onto the spuds.

**3.5 Shifting the platform to next location :**

a) Lower the platform onto water.

b) Hook-up a system of pulleys and wire ropes (L+B) on the short-posts

(fixed on Platform deck). Connect the lifting ropes from the spud-shoes.

c) Raise the spuds in a symmetrical fashion using the 2 T capacity winches fixed on the deck

d) When the spud-shoes are about 2 m above the seabed, lock them in position onto the platform.

e) Using a powered boat, lift the marine anchors, one by one.

f) Shift the platform, using the powered boat and anchor ropes, to manoeuvre it near the next borehole location.

g) Using the Theodolites, on the shore, mark the position of the next borehole and guide the power boat to place and anchor a floating buoy on it.

h) Then follow the steps as in (3.4) g & h.

**4) MOST IMPORTANT**

When not in USE, NEVER, Keep the platform floating on the sea water, even if marine anchors are in position. High spuds, extending above the platform deck, make it highly unstable and the marine anchors can easily get loosened. In fact, unfortunately this is what happened after finishing the last borehole.

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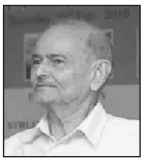

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# Aakaar 2012

Aakaar is the annual technical festival of the Department of Civil Engineering, Indian Institute of Technology (IIT) Bombay. The fourth edition of this festival is scheduled to be held on the 3rd and 4th of March, 2012.

In this edition, Aakaar has taken the initiative to "Go green" in order to Spread environmental awareness.

The prime crowd-puller of this festival is a National Students' Symposium which would be conducted on the lines of a student conference. This event will see the participation of students from over 140 colleges across India presenting their research work. "The Symposium has been instituted with the aim of providing budding civil engineers a modest pedestal to put forth their ideas, be heard on a grand platform and get assessed by popular industry professionals and adept professors. To add to this manifold undertaking, a multitude of competitions, technical workshops and guest lectures by veterans in the field of civil and environmental engineering would also be organized during the festival.

Prominent companies from the civil engineering sector get associated with Aakaar through events like the Lecture series and Panel debates.

This free event is open to all Civil Engineering students.

If you are interested in attending this event please contact Jaymin Kotecha on: +91 9420662743 or email:jaymin.k.iitian@iitb.ac.in

More information is available on the Aakaar website:www.aakaariitb.org

# METHODS OF STRUCTURAL ANALYSIS: UTILITY OF FINITE ELEMENT METHOD REVISITED

**Dr. M. G. Shaikh**

## 1. Introduction

The actual behavior of various structures under different circumstances of loading has been always a sort of mystery for engineers. The mathematical depiction of real behavior of structures in toto as it is, is a difficult task. The engineers have, therefore, found a way of idealizations of the real structures for getting some useful information about the structural behavior. In case of the idealized structures, the mathematical formulations became possible. Amongst such formulations, a few of them have exact analytical solutions in mathematical sense, and others do not have. This mathematical exactness should not be confused with the actual natural behavior of structures. In case there is no exact solution available due to complexities in the formulation, engineers look for approximate yet useful solutions. In literature, there are many approximate methods available. The methods in which some approximation polynomials along with appropriate multiplying parameters are used are known as variational methods. These methods include Rayleigh-Ritz method, least squares method, collocation method, etc. These methods differ in use of different functions as weight functions. They are applied for the entire domain of the problem. A method similar to Rayleigh-Ritz method applied to different sub-regions of given domain is known as finite element method.

Now, for the sake of analysis, structures can be grouped in two broad categories: discrete structures and continuum structures. The discrete type of structures consist of various structural elements are connected at their ends to form jointed structures, whereas in continuum type structures, it is assembly of structural elements joined together along their surfaces.

## 2. Analysis of Discrete Structures

Amongst discrete type of structures, some of them are determinate while others are indeterminate type. The analysis of determinate structures is quite easy where the equations of equilibrium (laws of statics) are used to find unknown quantities in the given problem. However, the analysis of indeterminate structures becomes difficult due to more number of unknowns than the available number of equations. The additional required number of equations in such cases is obtained by considering conditions of compatibility in the structure. The indeterminate structures can be analyzed by classical methods and matrix methods available in literature. In the earlier days when computers were not available, classical methods were in vogue. With advent of computers, the matrix methods proved very useful. One main difference between the classical methods and matrix methods is in the approach they have: classical methods consider structure as a whole (system approach), while matrix methods consider structure as assembly of constituent elements (element approach). Of course, computer programming is possible for classical methods too.

In matrix methods, depending upon the unknowns considered in analysis, they are categorized as flexibility methods and stiffness methods. It is interesting to see that even classical methods could be classified either as a flexibility method or as a stiffness method, albeit with system approach. In flexibility methods, forces in the structure system are taken as unknowns. The examples of such methods are method of consistent deformations, three-moment equation, strain energy method, column analogy method, influence coefficient method, etc. In stiffness methods, displacements in the structure system are taken as unknowns. The examples of such methods are slope-deflection method, moment distribution method, direct stiffness method. The flexibility methods are also known as force method or compatibility method, whereas stiffness methods are also known as displacement method or equilibrium method.

In flexibility based method, given indeterminate structure is reduced to a stable statically determinate structure by removing some appropriate constraints at supports of the structure. This is act of releasing the structure from redundant forces. Then unit forces are applied on the determinate structure and deformations (slopes and deflections) at the points of application of each redundant are determined either by moment-area method, conjugate beam method, or unit load method. Here, by imposing the conditions of compatibility in the original indeterminate structure, as many equations as the number of redundant forces in the structure are obtained. Then the equations so formed are solved by forming flexibility matrix and redundant force vector from these equations.

The degree of kinematic indeterminacy of given structure represents the unrestrained displacements at supports. This gives number of equations needed to find the unknown displacements in the problem. In stiffness based method, the given indeterminate structure is reduced to a kinematically determinate structure by providing appropriate restraints at supports of the structure. For example, in a continuous beam like structure, essentially we apply moments at the supports to have zero rotations at the intermediate supports of the beam. Then we look for the forces and moments required for unit rotation at a support while maintaining zero rotation at other supports. This process is repeated at all supports corresponding to the kinematic degrees of freedom. Then, since in the given structure, there were zero moments applied externally, the equilibrium equation, consisting of algebraic sum of fixed end moment and sum of products of stiffness coefficients and expected rotations at respective supports, can be written. Then the equations so formed are solved by forming equivalent nodal load vector due to actual applied loads (fixed end moments), stiffness matrix and unknown displacement vector from these equations.

Thus, it can be seen that the number of equations in flexibility method is equal to the number of redundants, while, the number of equations in the displacement method is equal to the number of constraints required to make the displacements zero. That is, the size of flexibility matrix is equal to degree of static indeterminacy, and that of stiffness matrix is equal to degree of kinematic indeterminacy. Smaller matrix means lesser number of equations to be solved. Therefore, by comparing size of these two matrices, suitable method for a given problem can be decided.

**3. Analysis of Continuum Structures**

In continuum structures, we do not have discrete structural members which could be joined together to form the given structure. Therefore, the methods mentioned above, which are good for analysis of discrete type structures, are not useful for analysis of continuum type structures. In case of such continuum structures, a numerical method known as finite element method, mentioned at end of the first paragraph, is found quite useful. Besides finite element method, there are a few other methods also available for analysis of continuum type structures.

In finite element method, which is a systematic numerical procedure, the given structure is divided into a number of small finite regular shaped subregions. The geometric region of the structure under consideration is known mathematically as domain, and the process of dividing it into number of subregions is called discretization. These subregions are called as finite elements. The finite word is used to distinguish them from infinitesimal elements used in derivation of governing differential equations. Such finite elements are considered connected to each other only at their vertices. The points in the given domain where the selected number of elements are connected with each other are known as nodes. An element may have some nodes in addition to its nodes at vertices. It depends on the degree of approximation desired in the analysis. However, these additional nodes do not connect adjoining elements with each other. For example, if given structure is discretized using triangular elements, one may use either triangular elements with three nodes at vertices only, or triangular elements with three nodes at vertices and three more nodes at mid or so on its three sides. The latter type of element is known as higher order element. The discretization of structure gives so called finite element mesh; if size, shape and type of every element is same, it is known as uniform mesh, otherwise nonuniform.

After discretization of given structure into a preselected type of elements, properties of elements are obtained. To do that, for example, if we are interested in displacement at various points in the continuum, the displacement at a point is approximated as a polynomial in terms of nodal displacement values and some appropriate so called shape functions. Now, if it is a one-dimensional problem, the displacements to be calculated are unidirectional like displacements in x-direction. In case of two-dimensional problem, the desired displacements at various points will have two components like displacement along x and y directions. Similarly, in three-dimensional problems, there will be three components of displacement at a point viz. components along x, y, and z directions respectively. These possible components of displacements are called as degrees of freedom. The number of shape functions required in the approximation polynomial mentioned above is equal to the total number of degrees of freedom of the element being used. These shape functions do possess properties like every shape function has its value equal to one at the corresponding node while its value at every other node is zero; another property is that the sum of values of all shape functions at any given point in the element is again one. Now every element can be described in global reference system as well as local reference system of individual element. Therefore the shape functions also could be written in global as well as local coordinates for each of the elements. These shape functions could be, depending on degree of approximation polynomial adopted in an attempt of structural analysis, either linear, quadratic, or of higher orders. These shape functions are then used as weight functions (Galerkin method) one-by-one in the weighted-integral form of the given differential equation and a set of algebraic equations is obtained. The number of equations in this set is equal to total number of degrees of freedom of an element. Thus, if there are only two nodal degrees of freedom as in case of a bar element, the number of equations mentioned above would be only two. These simultaneous algebraic equations are then transformed in matrix form to obtain stiffness matrix, displacement vector and force vector. Out of these three entities, the stiffness matrix is elemental structure property which is completely known for the given structure. In the force vector, some are known nodal forces (externally applied ones) while some are unknown ones (internal forces). Similarly, in the displacement vector, some are known because of knowledge of boundary conditions, while others are unknown. With proper condensation of these matrices, the unknown displacements at various nodes, and hence unknown internal forces, can be calculated. The other structural responses like strains, stresses, and forces can be derived from the knowledge of displacements at various nodes in the problem, using constitution relationships, and preliminary principles of mechanics. The results obtained by finite element method can be represented in tabular or graphical form.

The advantages of finite element method could be noted as follows:

1. Since a finite element solution consists of a set of algebraic equations rather than that of differential equations or integral expressions, finite element method is a preferred method of analysis for continuum type structures.
2. Different parts of a given structure can be discretized using different types of elements depending on corresponding structural actions like bar action, beam action, plate action, etc. in a single analysis.
3. Accuracy of structural analysis can be increased by using higher order elements with their proper density in the domain under consideration.
4. The shape functions for various types of finite elements can be obtained in a systematic way.
5. The curved or irregular boundaries of stressed bodies can be approximated closely by straight lines of suitable smaller lengths; finite elements with curved boundaries are also available in literature.
6. Structures with any size and shape of holes also could be discretized using suitable density of preselected elements to take care of stress concentration around such holes.
7. It is possible to convert any given loading like point load, distributed load, thermal load, prestressing force, etc. into nodal loads and use in the method. Various types of loads like electromechanical loads also could be considered simultaneously with knowledge of proper constitutive relations.
8. Structural bodies made by different materials in composite form also can be analyzed. These materials could be isotropic, orthotropic, or even anisotropic.
9. Structural problems where deformations are expected in different regimes like linear-nonlinear, static-dynamic, or even elastic-plastic can be analyzed using the finite element method.

**4. A Word of Caution**

Computers are a boon for structural engineers. Huge volumes of calculations are possible at amazing speed. However, the computers should be used with due care. Computers perform the task they are instructed to do. The proper set of instructions prepared by a structural engineer depends totally on his/her qualitative as well as quantitative understanding of the problem in hand. The input data fed to the computer should represent the variables involved in the physical problem as closely as possible. Otherwise the rule of 'garbage in - garbage out' works without fail! Moreover, interpretation of the computer output is equally important to arrive at proper understanding of the structural analysis in hand.

**5. References:**

Cook, R. D., Malkus, D. S., Plesha, M. E. and Witt, R. J., Concepts and Applications of Finite Element Analysis, 4th Edition, John Wiley & Sons, New York

Jain, Ashok K., Advanced Structural Analysis with Finite Element and Computer Applications, 2nd Edition, Nem Chand & Bros, Roorkee, U.A., India

Reddy, J. N., An Introduction to Finite Element Method, 2nd Edition, McGraw Hills INC New York 1993 Publication

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# Critical Review of the IS:875 (Part 3) – 1987 : Wind Loads

**K. Suresh Kumar**

## 1 INTRODUCTION

The Indian standard of practice (IS:875 Part 3, 1987) is widely used for estimating wind loads by practitioners in India. Prem Krishna (2006) and Subhash and Tamura (2007) discussed the need for an immediate review of the IS:875 and recommended further investigations. Based on our comparison of tunnel results with the codes, the IS:875 to our surprise stood apart from other codes and quite differently from the wind tunnel results. This led us to embark on a test project to compare the IS:875 predictions with the wind tunnel results for a standard building. The widely tested CAARC building model (Melbourne, 1980) has been chosen for this study and the details of the experiments can be found in Suresh Kumar (2011). Considering that the standards are based on simple box-like structures without any immediate surroundings, it is expected the wind tunnel results of CAARC model in line with values from codes of practice. This article presents both cladding and structural load comparisons between wind tunnel tests and IS:875 including the predictions from other international standards. Note that only vertical facades have been considered for the local wind loads considering the significant usage of glazing.

## 2 LOCAL WIND LOADS

To bring the wind tunnel results in similar format with the IS and ASCE codes, the wind tunnel results have been converted to pressure coefficients based on peak dynamic pressure at roof height. These results have been plotted in Figure 1 for selected elevations at corresponding tap locations for negative and positive pressures. The provided pressure coefficients in Figure 1 are minimum and maximum pressure coefficients at individual tap locations irrespective of the angle of attack.

Negative pressure coefficients are higher at the edges and moderate at the center as expected from typical flow regime. However, in case of positive pressure coefficients, they seem to increase along the height with the exception of extreme top region where they reduce as the flow tries to escape. Along with the wind tunnel results, the pressure coefficients from ASCE7-05 (2005), NBC2005 (2005) and IS:875 are also provided at the top. Note that equivalent NBC pressure coefficients in the same format as those with ASCE and IS have been derived and presented in this plot.

As far as the negative pressure coefficients are concerned, IS:875 value is -1.2 and this code does not distinguish between edge and center zones. This coefficient is much lower than the edge zone pressure coefficients obtained from wind tunnel results. Also, ASCE and NBC codes provide higher values as well for edge zones. However, IS and NBC values for center zone seem closer to the wind tunnel predictions. Note that certain variations in pressure coefficients can be attributed to the conversions from one format to the other. Overall, we can infer that the IS:875 significantly underestimate loads on claddings located especially on the edges. Also it appears that some of the local pressure coefficient data on the standard could have been obtained under smooth flow conditions and this may be the reason for a lower value. As far as the positive pressure coefficients are concerned, the wind tunnel results are in good match with ASCE and NBC values, while the IS:875 is not providing any value.

Finally, an example has been worked out for predicting local cladding load on a CAARC type building in Mumbai using wind tunnel and various codes. The basic wind velocity for Mumbai of 44 m/s 3-sec gust has been used for this exercise. Figure 2 shows the comparison of peak negative pressures along the height of the building. Note that the wind tunnel predicted cladding loads are between ASCE and NBC predictions, while the IS code predictions are far below. When the average wind tunnel predicted pressure is -3.75 kPa, the maximum IS prediction is only -2.3 kPa. This result shows that the IS code under-predicts cladding loads significantly and the code should be used with caution. Similarly, the positive pressure distributions have been compared in Figure 2. Note that the wind tunnel predictions are between the ASCE and NBC codes. However, the IS code does not provide any positive pressure value.

In a recent three tower project (heights of the order of 350m) in Mumbai, the IS:875 prediction of the external pressure was -2.5kPa, while ASCE7-05 predictions were -2.5kPa & -4.5kPa. The wind tunnel predictions went up to -6kPa though these high values were hot-spots at isolated locations on the façade. The more typical wind tunnel values on majority of the façade were within the ASCE predictions, but much higher than the IS:875 value.

## 3 OVERALL WIND LOADS

For the analysis, full-scale 3-sec gust wind speed of 44 m/s at 10 m height in open terrain is used. The raw overall base moments (maximum, mean and minimum) obtained from the tunnel for each wind direction are plotted in Figure 3. It can be noticed that there is across-wind loading at angles 90° and 270° in the X direction (see My), while the across-wind loading is dominating in the Y direction for wind angles 0° and 180° (see Mx). Considering the aspect ratio, as expected the across-wind effects are significant along the Y-dir when the wind is blowing perpendicular to the wider face or along X-dir. Winds blowing along Y-dir also induce across-wind loading in the X-dir but only as high as the along-wind loading. Note that mean loading will peak at the along-wind

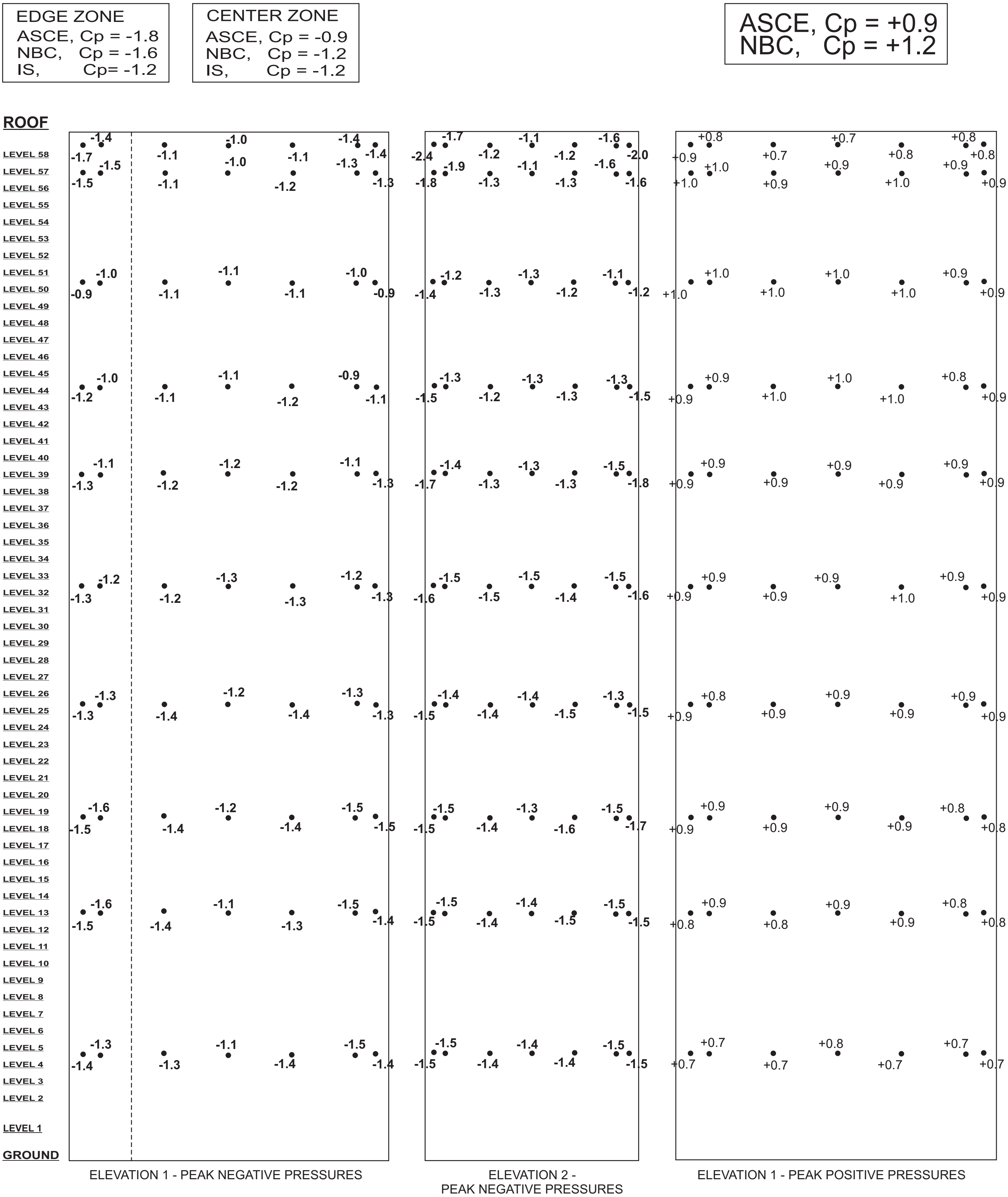



Figure 1. Peak pressure coefficients on CAARC facades.

loading condition in comparison to the zero or minimum mean value at the across-wind loading condition due to the symmetry of the flow.

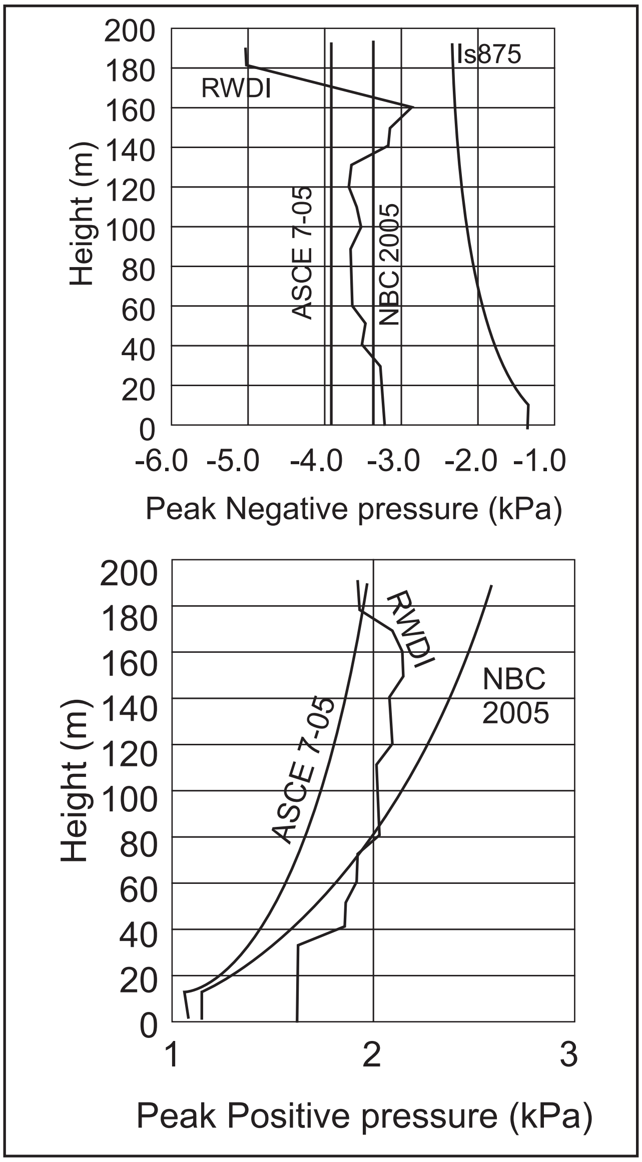



Figure 2. Comparison of peak pressures on CAARC building.

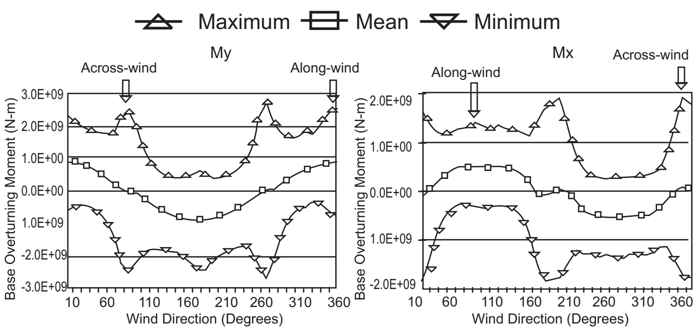



Figure 3. Raw overall base moments on CAARC building.

Figure 3. Raw overall base moments on CAARC building.

The overall peak base loads calculated analytically using the IS:875 is compared against ASCE and the wind tunnel values, and they are presented in Table 1. The results show that the values predicted in the X direction using IS:875 are generally in good agreement with the tunnel values, while the ASCE values are lower. In the Y-direction, both IS and ASCE are predicting lower values than the tunnel values. It is observed from Figure 3 that the peak along-wind loading compares well with the code predicted values for both directions. This shows that the tunnel predicted peak values are based on across-wind loading when both codes are predicting along-wind loads. In summary, the codes will under predict the loads in cases where across-wind phenomenon is dominating.

**Table 1. Overall base loads on CAARC building.**

| | Fx (kN) | Fy (kN) | My (kN-m) | Mx (kN-m) | Mz (kN-m) |
|---|---|---|---|---|---|
| **IS** | 2.39E+04 | 1.54E+04 | 2.42E+06 | 1.56E+06 | - |
| **ASCE** | 2.27E+04 | 1.41E+04 | 2.18E+06 | 1.36E+06 | 1.57E+05 |
| **RWDI** | 2.39E+04 | 1.64E+04 | 2.66E+06 | 1.93E+06 | 1.55E+05 |

At all angles of attack, buildings will be subjected to simultaneous action of sway loading at orthogonal directions as well as torsional loading. But the torsional loading didn't get the needed attention in many international codes of practice similar to IS code. In case of taller/slender buildings with lack of torsional stiffness caused by inefficient structural systems, torsional response/loading becomes even more important in comparison to the short/stiff building. Figure 4 shows the torsional loading measured on the CAARC building. Note that sharp flow separations at skewed angles ($90^\circ$ & $270^\circ$) of flow against the narrow face induce peak torsional forces on the building. In a square building scenario, peak torsional loading of equal magnitude will occur at four locations at skewed angles of flow against the four faces. In this case, the peak torsional moment is normalized by the maximum shear force as well as width to get the torsional eccentricity factor (e). The torsional eccentricity factor (e) for this case is about 15%. Based on numerous such studies in our lab and elsewhere, the torsional eccentricity factor typically ranges between 5% and 25% depending on the geometry and the surroundings. For preliminary estimation purposes, a torsional eccentricity factor (e) of 15% can be used similar to ASCE code. The torsional moment (Mz) at floor levels can be calculated by multiplying the maximum shear (F) with maximum width (B) and 15% torsional eccentricity factor; i.e. $Mz = F \times B \times 0.15$.

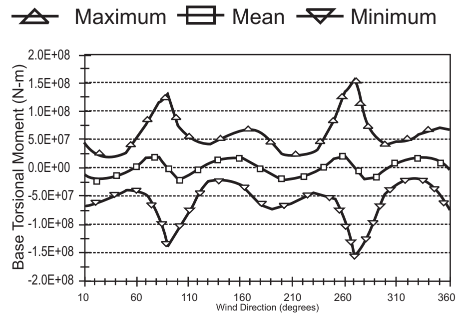


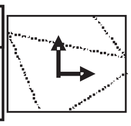
Figure 4. Overall trsional moment on CAARC building.

Figure 4. Overall torsional moment on CAARC building.

In a recent tall building project (height~250m) in Mumbai, wind-induced loading on orthogonal directions were higher than the IS code predictions (see Table 2). Significant difference in the Y-direction load was due to across-wind phenomenon caused by peculiar geometry.

Table 2. Overall base loads on a tall building project in Mumbai.

| | Fx (kN) | Fy (kN) | My (kN-m) | Mx (kN-m) |
|---|---|---|---|---|
| IS | 2.43E+07 | 3.81E+07 | 3.59E+09 | 5.40E+09 |
| RWDI | 3.34E+07 | 9.75E+07 | 5.46E+09 | 1.69E+10 |


PLAN

In another project in Mumbai, structural engineers didn't give any consideration for simultaneous action of sway forces in orthogonal directions and torsional forces in their preliminary design based on IS:875. For this particular project, the wind tunnel results were lower than the individual code derived forces and as a result the team was happy initially before they realized about the simultaneous action of the three forces and the associated load combinations. Once the simultaneous action of the loads is accounted, then the resultant forces were higher than the individual code derived forces. We have noticed that this is particularly happened since there is no recommendation or not even a wording on this issue anywhere in IS:875. Considering the importance of this matter, at least provisions made in other international codes (ASCE7-05, 2005; NBC2005, 2005) should be considered at the earliest.

## 4. RECOMMENDATIONS

Based on this study, the following recommendations have been made for practitioners while using the IS:875 code for wind load calculations.

It is recommend the local pressure coefficient to be increased to -1.8 from the current value of -1.2 at least for an edge zone of 20% building width. For the central region, one can use the same pressure coefficient of -1.2 currently in the code. The practitioners can use the load calculation procedure in IS:875 with minimum risk (r=0.63, life of 50 years) by using the above recommended pressure coefficients.

Cross-wind effects should be assessed using other international codes of practice until such procedure shows up in the next revision of IS. Torsional loading shall be calculated based on maximum shear force, maximum width and a torsional eccentricity factor of 15% (Mz = F x B x 0.15). Based on our wide experience in the subject matter, for simultaneous application of loads, 100% of any individual primary force (Fx, Fy or Mz) can be considered with 60% of secondary forces (Fx, Fy or Mz).

## 5. CONCLUDING REMARKS

The results of this study indicate that concerning the cladding loads predicted by IS:875, the pressure values on edges stand much lower than the tunnel tested and ASCE values. Therefore, IS:875 predictions for cladding design should be used with caution and it is strongly recommended to look at other international codes at the preliminary design stage. Also, it is necessary to revamp the tabular values for external pressure coefficients provided for local cladding design at the earliest.

The structural load comparisons indicate that IS:875 is indeed good enough for preliminary structural load predictions in the absence of across-wind response domination. However, the necessity of including a basic estimation procedure for across-wind response is clear which will help practitioners accounting this phenomenon in early stage of the design itself. Provision for torsional loading should also be included in the code. Further, simultaneous application of sway forces and torsional loads with suggested combination factors shall be included as well.

This article mainly addressed the local wind loads on vertical facades as well as overall structural loads on tall buildings. Many other subject matter from the IS:875 standard such as wind speed map, loading on roof and other structures requires revision as well. There are number of typographical errors in various tables and figures and the user needs to review the selected values thoroughly before using it in the current format. In summary, it is vital that the review and update of the existing provision (IS:875 Part 3, 1987) shall be carried out at the earliest.

## 6. REFERENCES

1. ASCE7-05, 2005. Minimum Design Loads for Buildings and Other Structures. Structural Engineering Institute of ASCE, Reston, VA.
2. Dragoiescu, C., Garber, J., Suresh Kumar, K., 2006. A Comparison of force balance and pressure integration techniques for predicting wind-induced responses of tall buildings, in: Proceedings of the ASCE Structures Congress 2006 – Extreme Wind Engineering, St.Louis, USA.
3. IS:875 Part 3, 1987. Code of Practice for Design Loads (Other than Earthquake) For Buildings and Structures, Part 3 Wind Loads. Bureau of Indian Standards, New Delhi.
4. Melbourne, W.H., 1980. Comparison of measurements on the CAARC standard tall building model in simulated model wind flows. Journal of Wind Engineering and Industrial Aerodynamics 6, 73-88.
5. NBC2005, 2005. User's Guide NBC – 2005, Structural Commentaries (Part 4). Canadian Commission on Building and Fire Codes, National Research Council of Canada.
6. Prem Krishna, 2006. The Indian wind loading standard: A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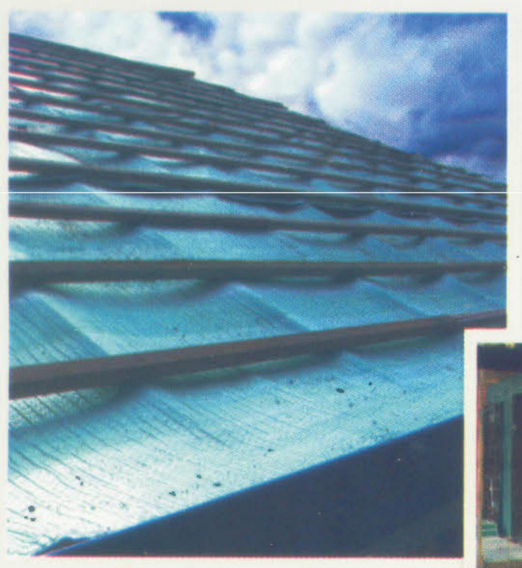

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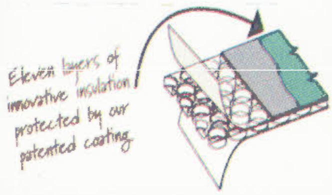


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