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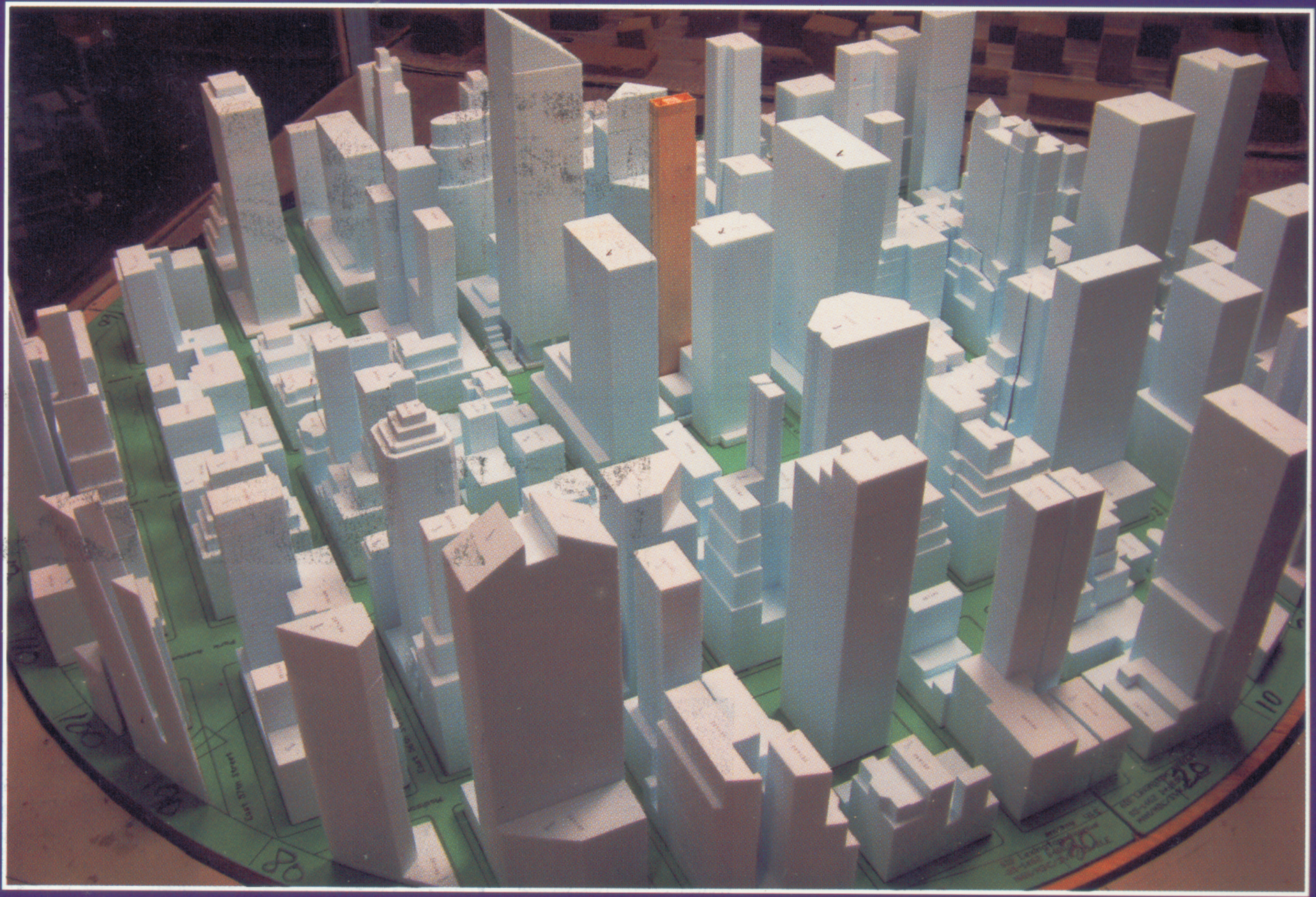

WIND TUNNEL TESTING

(See page 3 inside)

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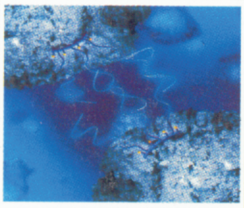

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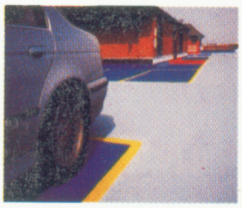

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Editor : **N K Bhattacharyya**
Jt. Editor : **Hemant Vadalkar**

AIMS & OBJECTIVES

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 organisations 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.

Fraternity News

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(Oct-Dec 2009)

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ADVANCEMENTS IN PREDICTION OF WIND LOADING ON TALL BUILDINGS

K. Suresh Kumar

ABSTRACT

Most of the existing international codes and standards for wind loading on structures clearly show their limitations and recommend detailed studies by other established means when required. As a result of this, in the past two decades, wind tunnels have been widely used to reliably predict the localized wind loads on the cladding/glazing as well as the overall structural wind loads on the frames of tall buildings. Similar to other codes/standards, the Indian wind loading standard also clearly shows its limitations and warrants detailed studies. Contrary to this, many high-rise buildings are being designed in India without carrying out detailed wind analysis. In many cases, the need for such detailed wind tunnel studies is apparent. This paper starts with a discussion on existing issues in India concerning wind engineered high-rise buildings. Thereafter, many wind tunnel studies carried out at RWDI will be utilized to show how a wind tunnel test can assist the designers regarding geometry change, surroundings, orientation, directionality, complex structural interaction, and cross-wind response.

1. INTRODUCTION

The importance of "Wind Engineering" is emerging in India ever since the need for taller and slender buildings came into picture. Considering the urbanization, limited space and sustainability, horizontal expansion is no more a viable solution especially in metropolitan cities like Mumbai, New Delhi etc. So all of this say that we have to grow up and for that sky is the limit! There is enough technology to build super-tall buildings today, but in India we are yet to catch up with the technology which is already established in other parts of the world.

Wind engineering is a niche field under the umbrella of Civil Engineering. Many times, wind engineering is being misunderstood as wind energy in India. On the other hand, wind engineering is unique part of engineering where the impact of wind on structures and its environment being studied. More specifically related to buildings, wind loads on claddings are required for the selection of the cladding systems and wind loads on the structural frames are required for the design of beams, columns, lateral bracing and foundations. Without doubt, the wind loads are combined along with other types of typical loads such as dead load and live loads using load combination procedures. Earthquake loads are another type of lateral load which is considered for design as well. However, considering the rare chance of simultaneous occurrence of both earthquake and high wind, both earthquake and wind loads won't be combined together in any structural design. As per our consulting experience, wind in general governs the design when buildings are above 150 m height. When buildings grow taller, they become flexible and they are moving away from the high frequency earthquake waves.

1.1 *History of Tall Buildings*

It is interesting to note that historically most of the high-rise buildings came up in North America before and majority of the 20th Century until up recently where there is a clear shift in momentum from North America to Middle-east/Asia. Also, the height of tall buildings was increasing through-out the last century at a steady pace until 2009 in which year the incremental change in height was substantial. These can be observed in Figure 1.

Figure 2 shows the tallest buildings completed in 2008 where most of the buildings are in Middle-east/Asia and this clearly shows the momentum shift from North America to Middle-east/Asia

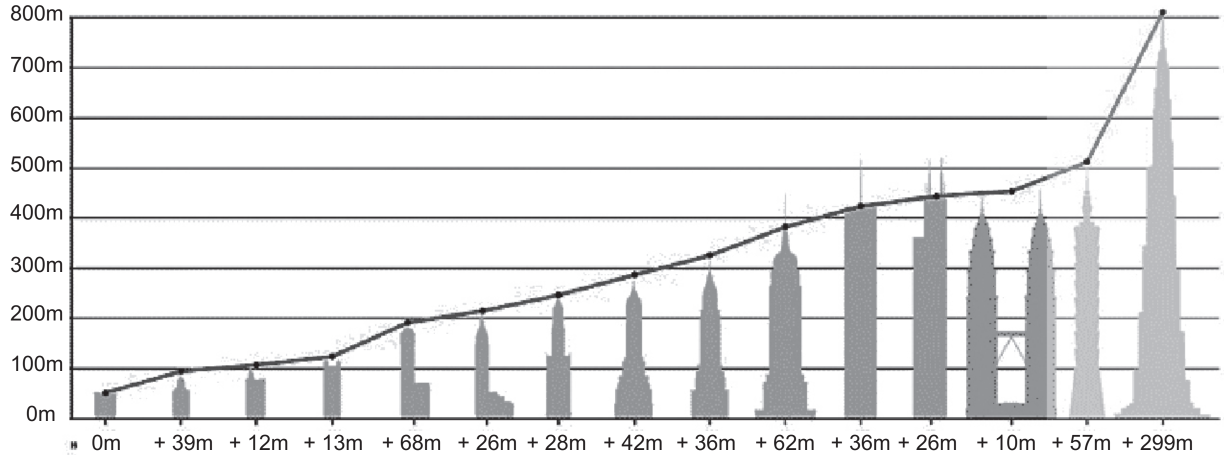

Figure 1: Height incremental changes in the development of the world's tallest building historically (copyright : (CBUH).

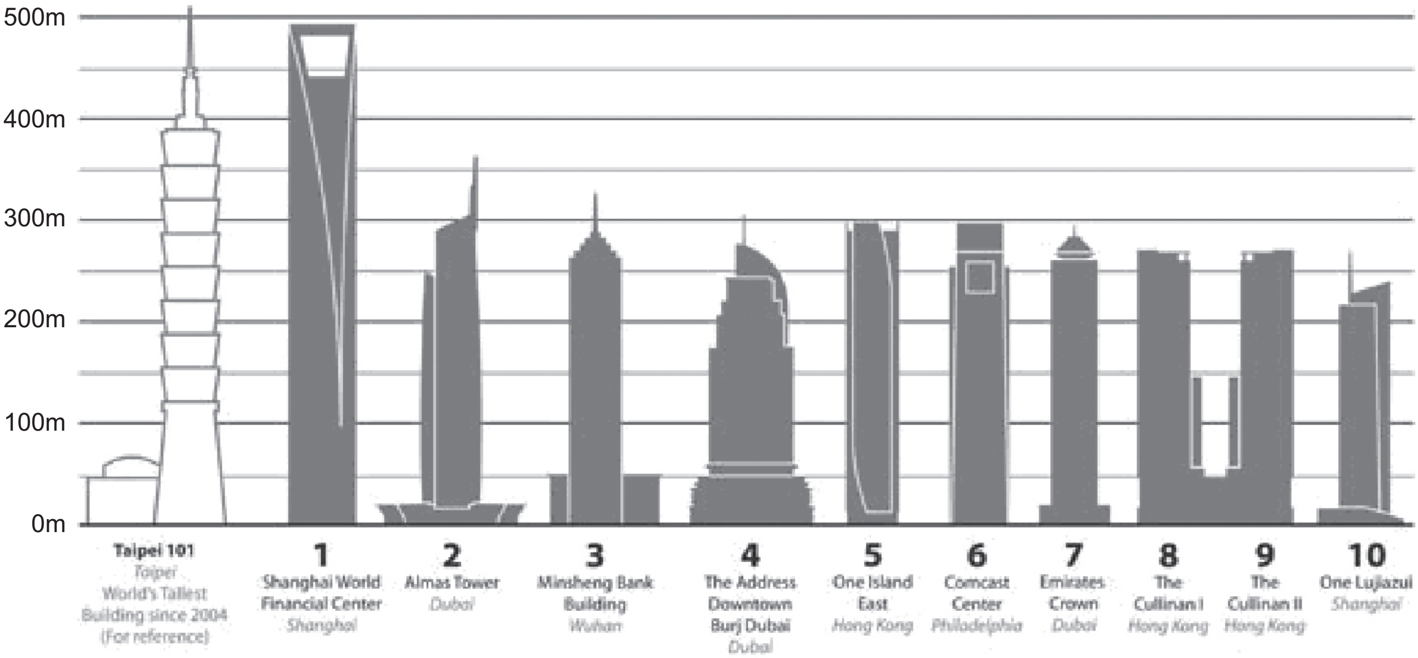

Figure 2 Tallest 10 buildings completed in 2008 (Copyright: CTBUH)

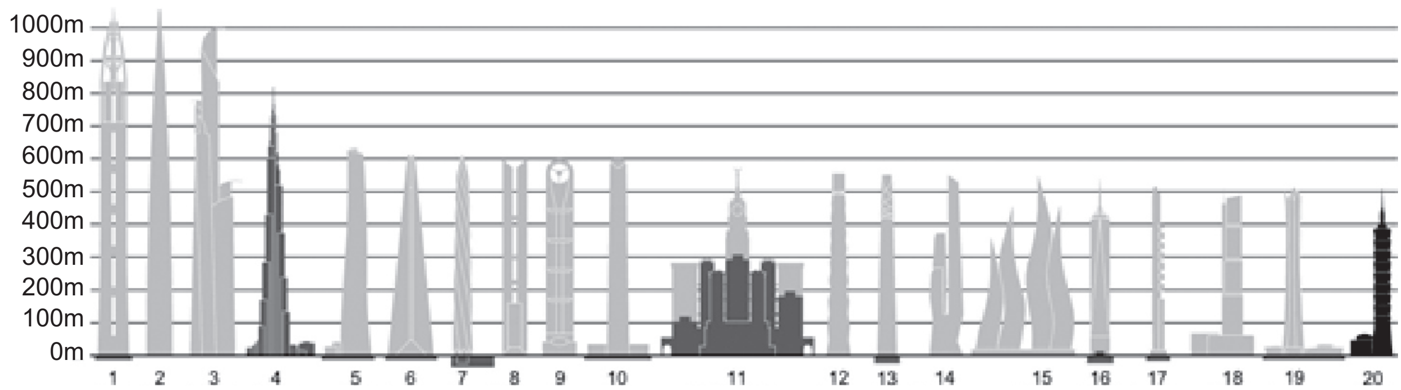

Figure 3 Tallest 20 buildings in 2020 (Copyright: CTBUH)

Recently CTBUH published the tallest 20 buildings in 2020 which is shown in Figure 3. Most of the buildings are in Middle-east/Asia region where Taipei 101 building (currently world's 2nd tallest building) sits at the 20th place.

1.2 *Wind Engineering Studies*

Wind engineering can be studied in many different ways and they are shown in Figure 4. They are briefed below:

Wind Tunnel: In wind tunnel studies, scaled models of structures are subjected to scaled atmospheric wind in a controlled laboratory set-up. Then sensors installed on the model can measure the physical quantities of interest such as shear, moment, pressure etc. Later in the analysis, these model scale quantities are converted to prototype using model scale laws. Most of the complex architectural and structural innovations are being confirmed through wind tunnel tests. Outside of India, wind tunnel tests are being done for almost all buildings above approximately 100 m. Even low buildings are being tested for places like Miami where severe wind conditions are expected.

Full-Scale: With full-scale studies, actual buildings already built will be used to get instrumented and put into test in natural wind flow for several months in order to get decent measurements. The time consumption in full-scale study is one of the issues; many times one has to wait for the wind to blow from the directions that you need since you don't have opportunity to rotate the building like in a wind tunnel. Full-scale studies are good to improve our understanding of the science as well as the simulation in wind tunnel. But these types of studies are not practical in day-to-day life.

Analytical: With analytical studies, the structures are modeled in structural dynamic sense and the wind flow is modeled as stochastic time series and thereafter, the response of the structure is obtained by random vibration techniques. This is a very useful tool in research since parametric studies can be easily carried out. For any analytical study, a few parameters have to be determined through wind tunnel tests.

CFD: In Computational Fluid Dynamics (CFD) studies, like in analytical studies, the structures are modeled in structural dynamic sense. But the wind flow is modeled using basic fluid dynamic equations such as continuity, energy and momentum equations. Thereafter a specific turbulence model is used and the equations are solved using some numerical techniques and the responses are obtained. This technique is becoming popular and quite widely used in studies such as pedestrian level wind flow, internal flows for air quality, pollution studies, topographical studies etc.

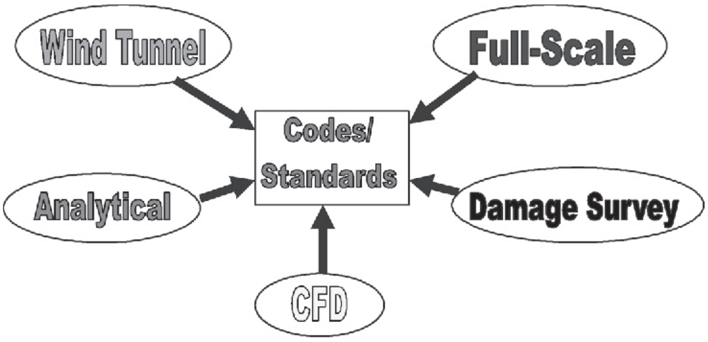

Figure 4 Different ways of wind engineering studies

Damage Survey: This is a rather new science emerged in the last two decades. Once the damage has occurred, pool of experts visit the place and try to investigate how it happened. Through this we try to relate the extent of damage with the wind speed. They may carry out some simple laboratory tests as a part of this.

Codes/Standards: Codes/standards are being widely used in India for determining wind loads for design. Certainly this is sufficient for preliminary design. However, considering the numerous non-typical geometries of the structures, complex surroundings and complex structural design which are not covered by the codes/standards, for final design other established means such as wind tunnel tests are absolutely required to confirm the preliminary design.

2. CODES / STANDARDS

Wind load/pressure information in codes/standards:

- Do not account the aerodynamic effect of the actual shape of the structure since they are based on box-like buildings
- Does not allow for any directional effects and as a result the design speed is assumed to be constant from all directions including the aerodynamically severe directions (blanket factors on some codes are exception)
- Assumes 'isolated' building condition without any influence of adjacent buildings

2.1 <u>Situation in India</u>

Currently, the general trend is to not do any special wind tunnel studies and the building design is simply based on the wind load provisions in the IS: 875 Standard. Even with the Standard, we have seen cases where the wind load code provisions are often misunderstood. This lack of understanding could be resolved easily by including a course addressing the issues of calculating dynamic lateral loads (i.e. wind, earthquake) using IS codes. We think this is essential considering the increasing number of high-rise buildings currently being proposed in India and RWDI would be pleased to help in this regard.

Another issue with the Standard is that the users believe that the code calculations provide the ultimate answer. Most users are unaware that the Standard is derived from a set of wind tunnel experiments of buildings with simple geometries. Like any other Standard/Code, the Indian Standard also states its limitations as follows:

*As per **IS:875 Part 3 1987**, "Note 1(Page 5) – This standard does not apply to buildings and structures with unconventional shapes, unusual locations, and abnormal environmental conditions that have not been covered in this code. Special investigations are necessary in such cases to establish wind loads and their effects. Wind tunnel studies may also be required in such situations. Note 2 (Page 5) – In the case of tall structures with unsymmetrical geometry, the designs may have to be checked for torsional effects due to wind pressure. Note 9 (Page 48) - In assessing wind loads due to such dynamic phenomenon as galloping, flutter and ovalling, the required information is not available either in the references of Note 8 or other literature, specialist advise shall be sought, including experiments on models in wind tunnels*

Unfortunately these limitations are shown in very small fonts at the beginning and towards the end of the IS:875 and hence maybe missed. We feel that these statements should have been in bold letters at the beginning of the document. We are sure that if this Standard were followed then we would end up getting more wind engineered buildings.

Note that a wind engineered building is not all about precise loads for design. One of the most important issues when the buildings grow higher is serviceability related to comfort. If the building users perceive motions, they don't like to stay and as a result, the building is considered substandard. At this stage of completion, it is quite difficult to correct a problem since the commonly used solution is to add damping through the use of a tuned mass damper or a tuned liquid column damper. These devices require space at the top of the building, which is generally not available. Serviceability is one of the serious issues, which all Standards/Codes are in general poor predictors. Furthermore, complications like modal coupling and cross-wind loads are rarely considered in codes. In addition, accounting for proper geometry, surroundings and directionality in wind tunnel tests is a big plus in identifying the precise loads acting on high-rise buildings and their performance. Often the wind tunnel test shows that the codes grossly overstate the necessary design load. Hence, significant savings are possible.

Also, we have noted that in India, common practice varies from that found in the international design communities. Some areas of concern are: (1) completing foundations before getting wind tunnel results, (2) if wind tunnel results are lower they wanted to use code loads anyway and (3) when wind tunnel tests are prescribed even though much simpler and cheaper methods are available, a very complicated aeroelastic model study in a wind tunnel is requested as the only approach to defining the design loads. All of these points should be challenged.

One other interesting issue is costs. Frequently when clients request a wind tunnel test, the first item for discussion is the cost related to the tests. They will try their best to reduce costs without knowing the benefit of doing such detailed studies. In many parts of the world, the costs of the wind engineering studies are only 10% of the savings that can be realized in just the structural system alone. In our opinion, costs should be secondary factor rather than the primary concern. On the other hand, in general, code derived loads are usually conservative and by doing wind tunnel studies you can generally reduce loads and save crores of rupees in construction costs. As we are now designing taller buildings with more challenging designs, we must move toward meeting international design standards. So whatever we can do to get there is the way to go!

3. WIND TUNNEL STUDY

In contrary to the codes/standards, wind tunnel testing

- Do physically simulate and predict the aerodynamic effect of the actual shape of the structure
- Do consider the directionality of the wind climate
- Do account the influence of adjacent structures by modeling the 'proximity' disk
- Do provide more refined and accurate loads

3.1 *Geometry*

The case study showed here deals with the wind-induced response of a 505 m tall Taipei 101 building, which is currently the world's second tallest building. Initial design had a square cross-section shape (UM), and later on during the wind tunnel test, significant cross-wind forces were observed. Thereafter, with a view to reduce the significant cross-wind loading on the building, various corner modifications such as angled, single step, double step and rounded corners were tested in RWDI's wind tunnel. It is noted that the double step corner (MC-1) provided the maximum reduction in the across-wind loading out of all the various corner configurations tested. Figure 5 shows the rms across-wind loading (Fy) and mean along wind loading (Fx) measured on this building for the original square configuration as well as the double step corner configuration, along with a picture of the completed tower. It is clear that significant reduction is possible in dynamic cross-wind loading by changing the square corner to a double step corner.

This reduction in loading is due to the disruption of severe vortex shedding by the corner modification. Note that usually the design of a building of this height and shape is driven by cross-wind loading and any reduction in cross-wind loading by changing the geometry of the cross-section will result in cost effective design. Further, it is also noted that double step corner reduced the mean along-wind loading.

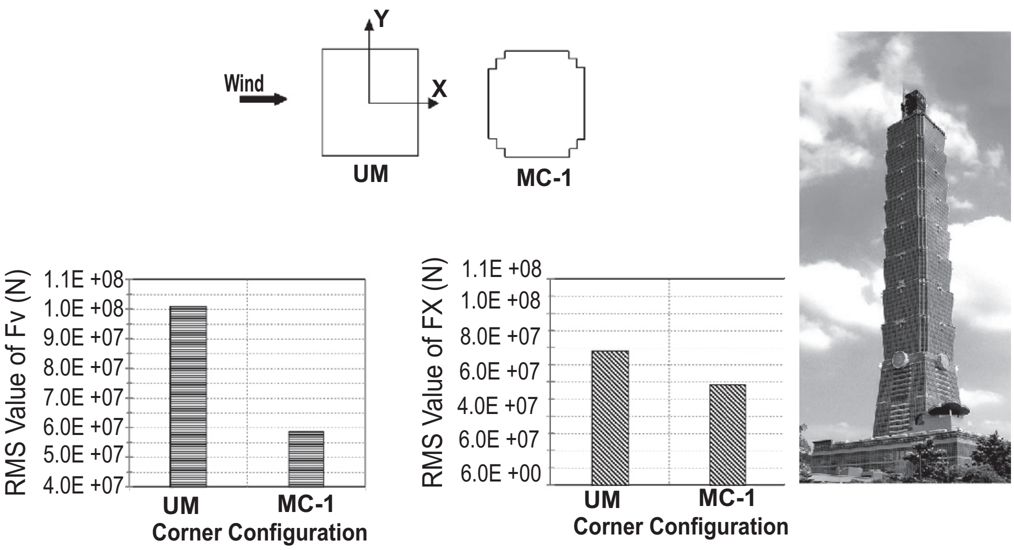

Figure 5 Effect of Geometry on Wind-Induced Response

Figure 6 shows complex roof geometry as well as a tall building façade tested in RWDI's wind tunnel and in such non-typical cases, wind tunnel tests are probably absolutely essential.

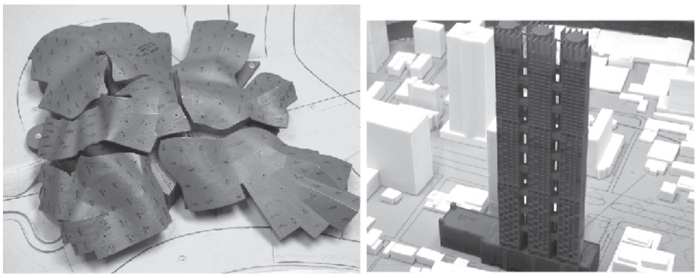

Figure 6 Examples of complex geometry wind tunnel model

Figure 7 shows couple of most recent air traffic control tower jobs. These types of towers are prone to wind-induced vibrations and a closer look at the geometry from the initial stage of the design would certainly help to reduce the motion issues. In case of the tower shown in the first photo, the structural engineer was in touch with us especially in initial stage of the design to make sure the geometry is appropriate to reduce motion issues. Through many discussions, they have finalized the geometry and finally the better motion performance of the tower has been confirmed through wind tunnel tests. In the case of the second tower, we found motion issues after the tests though the structural engineer and architect worked on a better shape.

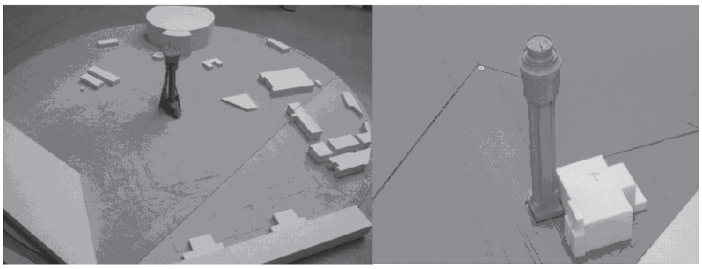

Figure 7 Wind tunnel models of Air Traffic Control towers

3.2 *Orientation and Directionality*

Wind tunnel testing can be a powerful tool in the architectural and structural design of tall buildings. Wind tunnel testing early in the design helps to minimize wind effects by reshaping the structure, fine-tuning stiffness and mass distributions and orienting the structure properly. The Burj Dubai Tower, recently completed in Dubai, UAE, is over 800 m in height and is currently the tallest building in the world. Figure 8 shows the roof plan, wind tunnel model and future view of this building. Early in the design of Burj Tower, several rounds of high-frequency-force-balance tests were undertaken and the resulting load data were used as early input for the structural design and subsequent parametric studies which optimize the stiffness and mass distributions.

Note that this building has six important wind directions, three of which are wind blowing into the nose and the other three are wind blows in between two of the wings, as shown in Figure 2. During this analysis, it was noted that the force spectra for different wind directions showed less excitation in the important frequency range for winds impacting the nose end of a wing than in-between the wings. This specific information was used to orient the structure in such a way that nose points to the most frequent strong wind directions in Dubai. Further, several rounds of tests carried out in this particular project helped to evolve the final geometry of the tower which defined the setbacks at different levels. The setbacks and tapering resulted in a substantial reduction in wind-induced response (Irwin & Baker, 2005).

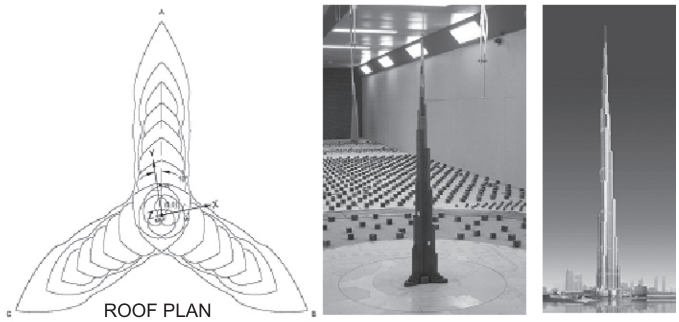

Figure 8 Roof plan, model scale and full-scale views of Burj Dubai Tower in Dubai

3.3 *Surroundings*

Structures often occur in groups. The response of one structure of such a group may be significantly altered due to the presence of the cluster. Vortices shedding into the flow by an upstream structure may cause Vortex-Induced Oscillation (VIO) like response, but at frequencies determined by the geometry of the upstream structure. On the other hand, a structure in the wake of another structure will be subjected to lower mean velocity producing a reduced drag. Note that flow interference is directionally sensitive; just small perturbations of the structure path or the direction of the incoming flow can result in dramatic changes. There are many parameters, which affect the manner in which one building modifies the forces on another building in its vicinity. These are size and shape of the building, wind velocity and direction, approach terrain and the geometry and proximity of neighboring buildings.

The results from the wind tunnel testing of an 85 m tall building (C) in an urban environment in Toronto, Canada are shown in Figure 9. In the vicinity of building C, there are other adjacent buildings taller than building C, as pictorially shown in Figure 9. The maximum, mean and minimum base torsional moments acting on building C with respect to wind direction are also shown in this figure. It is clear that for wind angles 220° through 240°, the base torsional moments grows to high values. Based on the location of the study building with respect to the surroundings, it is clear that this high loading is caused by the upstream building B and the distance between buildings B and C is about 2.5 times the width of building B. In order to confirm this, wind tunnel tests were repeated for these angles without the presence of building B and the results are shown. Without the presence of building B, the base torsional moment appears rather normal for an individual building. Similar results prevail for the case of sway moments.

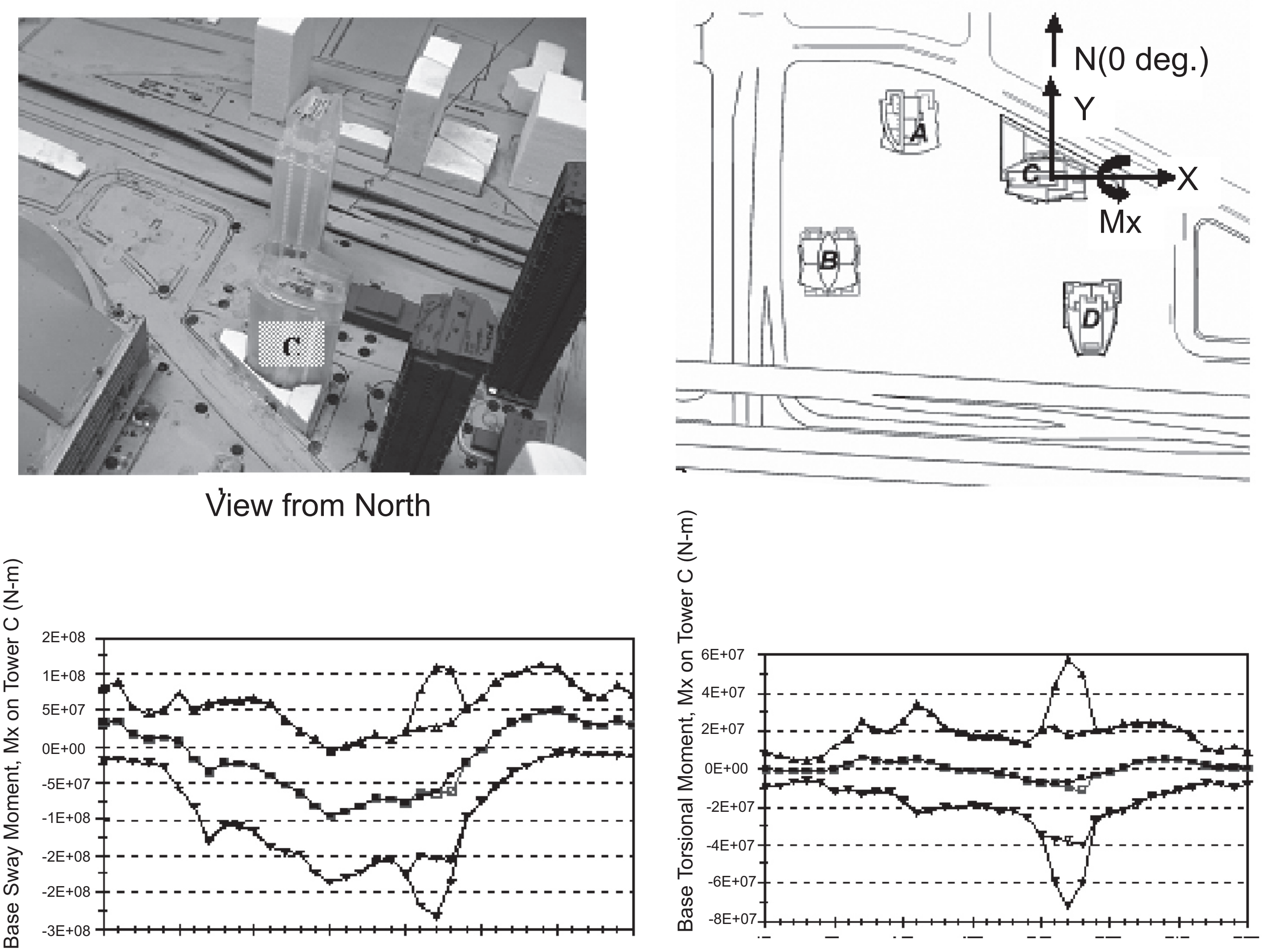

Figure 9 Effect of surroundings on an 85m tall building in Toronto, Canada

3.4 *Interaction Effects*

The study shown here is regarding the excessive wind-induced vibrations of rooftop features on a high-rise tower. These rooftop features are made up of an array of aluminum tubes supported by beams and columns, as shown in Figure 10. RWDI's objective was to define the failure mechanism and to provide conceptual solutions to the problem. This study is based on a review of the rooftop feature's drawings and video of the vibration condition, in-situ measurements of vibration frequencies and damping levels, a dynamic model of the roof features created by RWDI in SAP2000 and other desk-top calculations, and our engineering experience and judgement.

The observed wind-induced vibrations of the aluminum tubes were concluded to be due to the phenomenon of vortex excitation, compounded also by wake interaction effects between the tubes. The indications are that it is the first mode of vibration that is excited (with frequencies in the 10 to 12 Hz range). The first mode of vibration of the tubes was predicted to be excited in the speed range of 7 to 9 m/s at roof level, which is a daily event. The condition is accentuated by the very low damping of the tubes found in on-site measurements. Since the tubes are made of aluminum, which has low fatigue resistance, and since the wind speed range of 7 to 9 m/s is very common, it has not taken long for fatigue failures to occur. They have tended to occur where one would expect, i.e., at areas of stress concentration such as welds.

In summary, simple rooftop feature design using array of tubes turned out to be a nightmare and costly to fix. As an interim solution, RWDI recommended either filling the tubes with sand as it increases the mass and damping or tie all the tubes with a rope. Out of the two permanent conceptual solutions, the first one was to replace aluminum tubes with steel tubes. Due to the higher weight of steel, the oscillation would be smaller. Also, steel has much better fatigue resistance. Some form of visco-elastic pad between the steel tubes and their main supporting girders was also recommended. An alternative approach is to insert steel liners inside the aluminum tubes, as well as cross bracing the girders and add damping pads where the tubes are affixed to the girders.

It was noted in this project that the predicted wind loading in the x-direction, (i.e. narrow direction, My, Fx) is higher than normally expected. This is caused by the couplings of the x and y motions in the first two modes of vibration. As a sensitivity assessment, the wind loads were predicted without the modal coupling, and using the first mode frequency for both the x and y directions. From this uncoupled assessment, the x-direction wind loads were about 70% (factor of 0.7) of those resulted from coupled modes and the y-direction loads increased by about 8% (factor of 1.08). The primary way to reduce the x-direction loading in this case would be to either eliminate or reduce the coupling with Y by modifying the internal core. Oblique shear walls in the core can potentially uncouple the sways.

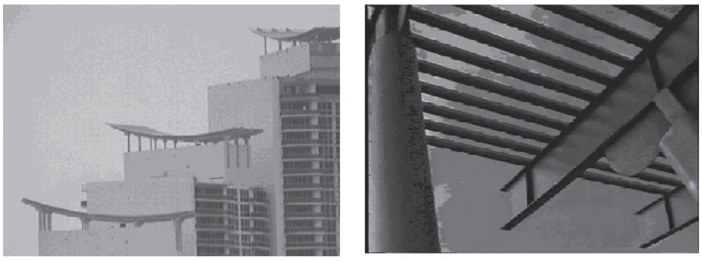

Figure 10 Tube array feature on roof top of a high-rise building

3.5 *Complex Structural Interaction*

Many times, complex structural properties would alone be the reason for higher wind-induced response. For instance, modal coupling is a common result of the complex structural design. There are different types of couplings and the typical one would be the coupling between torsion and sway modes and the unusual one is the coupling between sway modes and this could be eliminated by tweaking structural internal core. One such example is shown in Figure 11, where the wind tunnel tests were carried out for a 400 m tall Z-shaped building.

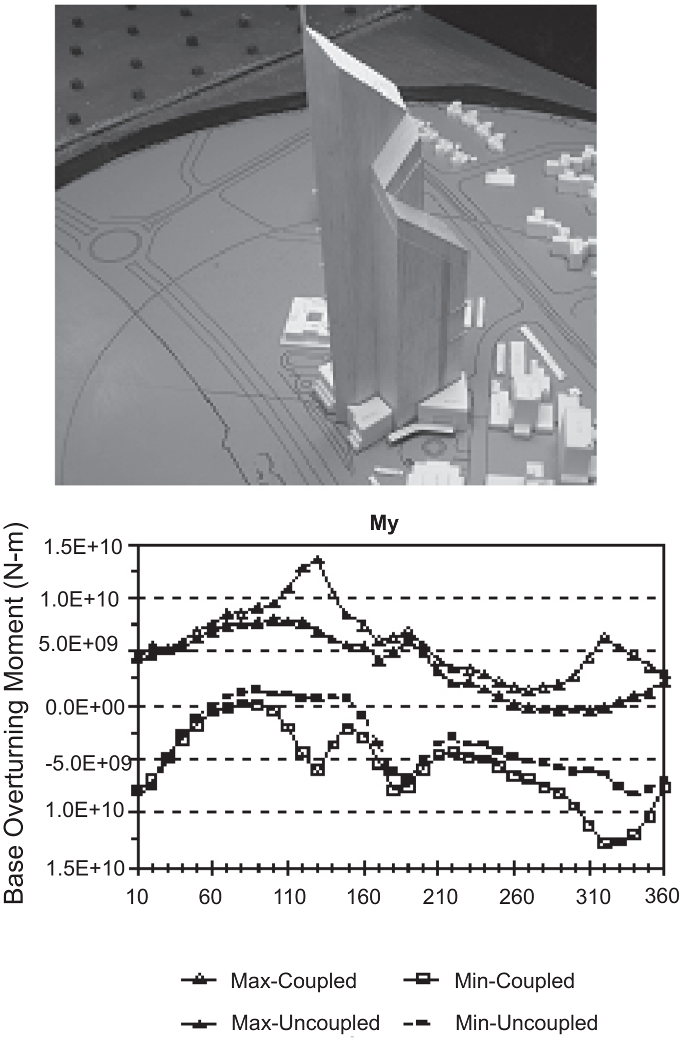

Figure 11 Effect of complex structural interaction

3.6 *Cross-Wind Response*

Cross-wind response of a building is mainly influenced by the geometry and immediate surroundings. The effects of these cannot be codified considering the numerous possible geometries and surroundings that we are encountering in real life. Therefore, site specific wind tunnel tests have to be carried out for determining loads on buildings.

Figure 12 shows the wind-induced base overturning moment of a 250 m high square building located in open surrounding condition. The cross-wind response is quite evident when wind blows normal to the face with approximately zero mean moment. Note that this building had fins and corner setbacks modifying the amount of cross wind response and the accelerations were within limits. Figure 13 shows the cross wind response of a typical rectangular shaped building with a height of 210 m. In contrary to the expectations, high cross-wind loads were not induced on this building as a result of crowded urban condition around this building, however the accelerations were high.

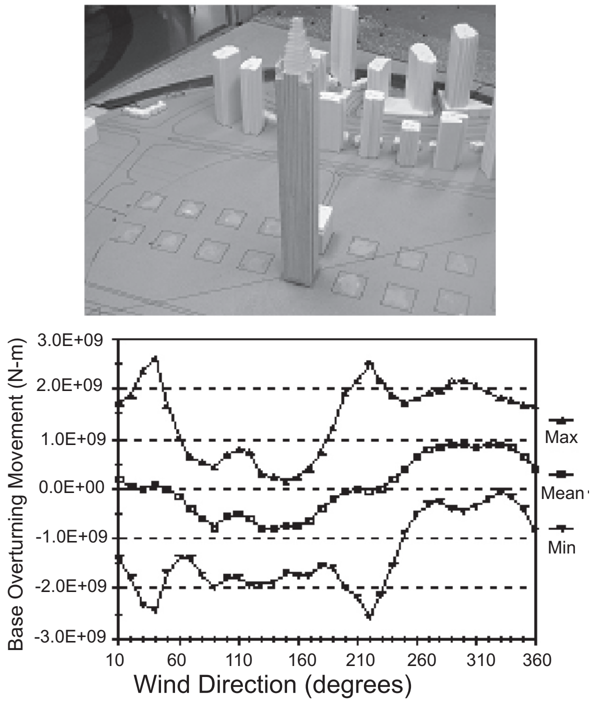

Figure 12 Cross wind response of square shaped building in open condition

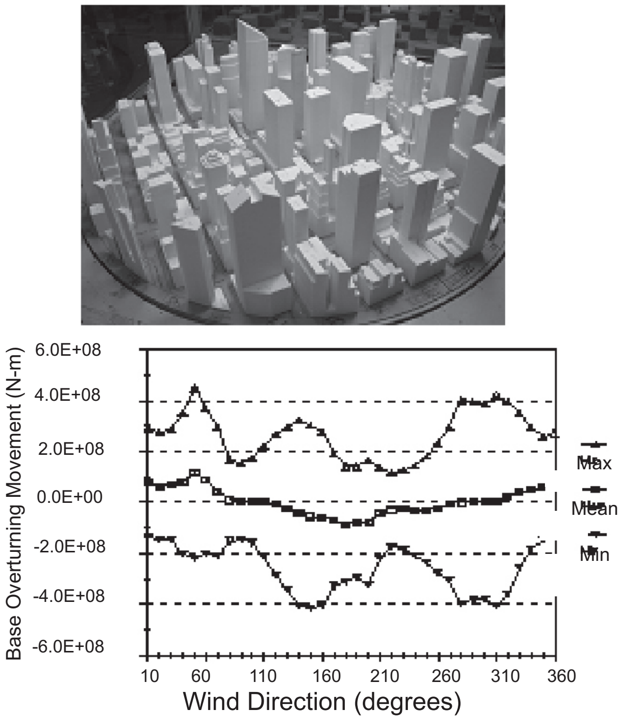

Figure 13 Cross wind response of rectangular building in urban condition

3.7 *Serviceability/Human Comfort*

All buildings are expected to move to some degree under wind action. When buildings grow taller and slender, such motions can be noticeable to their occupants and pose concerns. This happens when the magnitude of movement become significant and/or it's frequency of occurrence is excessive.

The wind tunnel tests on tall/slender structures have been carried out not just to determine the wind-induced loads but more importantly the wind-induced motions at top occupied level. The design of such structures demands the need of their motions to be restricted to within comfortable/acceptable limits. The motion perception can be quantified by maximum values velocity or acceleration. However, considering the physical quantity of acceleration's (rate of change of velocity) ability to induce force on human body as well as stimulate body organs, the acceleration parameter has been widely used to quantify building motions.

The Council on Tall Buildings and Urban Habitat (CTBUH) recommends 10-year peak resultant accelerations of 10-15 milli-g for residential buildings, 15-20 milli-g for hotels and 20-25 milli-g for office buildings (Isyumov, 1995). Generally, more stringent requirements are suggested for residential buildings (Irwin, 2004), which would have continuous occupancy in comparison to office buildings usually occupied only part of the time and whose occupants have the option of leaving the building in advance of a storm.

4. WHEN TO DO A TUNNEL TEST

Interestingly enough nowadays developers and architects are coming up with unconventional building shapes with offsets, setback, various corner shapes, balconies, fins etc. Further, the buildings are mostly located in complex surroundings along with other structures. These conditions were not covered or addressed in any of the international codes and standards. In addition to this, the effect of building response due to its orientation with respect to the wind directionality of the site is not covered in detail in any of the International codes. In general, Code analytical methods are helpful for preliminary design and for simple situations, but provide conservative wind loads in most cases; underestimating in others.

Further, complex structural interaction as well as cross wind response which are important for tall buildings are not covered generally in codes and standards. Many times, complex structural properties would alone be the reason for higher wind-induced response. On the other hand, cross-wind responses of buildings are mainly influenced by the geometry and immediate surroundings. The effects of these cannot be codified considering the numerous possible geometries and surroundings that we are encountering in real life. Therefore, site specific wind tunnel tests have to be carried out for determining loads on buildings.

Based on Indian wind conditions, it is advisable to carry out wind tunnel tests when buildings are above 30 storeys (or 100 m above grade). Slenderness ratio is another criterion to be considered when deciding wind tunnel tests. Wind tunnel tests are suggested when slenderness ratio (height/width) exceeds 5.

4.1 *Myth and Reality*

(1) Myth: Higher load provisions given by tunnel testing
Reality:
- Codal values are not always the upper bound
- Cladding loads in IS:875 are underestimated (Bhami et al., 2009)
- Tunnel results are the most accurate and have to be accounted
- 85% of the projects are lower than code (structural)

(2) Myth: Reluctance to use lower loads from tunnel test
Reality:
- Geometry and surroundings can potentially lower the loads
- Codes are based on simple geometry and no surroundings

5. GENERAL REMEDIAL MEASURES

In case of an issue with the wind-induced response, the remedial measures typically followed are described below.

5.1 *Aerodynamic Modifications*

Typically the geometry can be modified to reduce the wind-induced response though this is not a preferred method many times. For instance rounded corners or step corners on sharp edged cross sections can reduce the intensity of vortex shedding and can reduce cross-wind loading. Since vortex shedding and galloping are caused by the shape of the structure, modifications to the shape have the potential to eliminate the root cause of aerodynamic stability problems. Helical strakes, and porous or slatted shrouds installed at the ends of circular cylinders are effective measures to reduce the effect of vortex shedding.

5.2 *Structural Modifications*Raise Natural Frequency:

The natural frequency depends on the stiffness and mass of the structure. Thickening sections to increase stiffness leads however to increased mass, so that the end effect on frequency can be quite small. Thus, this is not a very practical approach. On the other hand, with appropriate positioning of the internal structure could raise the frequencies without adding mass. Note that the role of stiffness may sometimes be altered depending on whether loads or accelerations are important. For the case of vibrations caused by longitudinal turbulence, increasing stiffness is always beneficial to reduce loads and accelerations; however, in case of vortex shedding, increasing stiffness will worsen the situation unless the increase is sufficient to raise the critical speed to a value well beyond the design speed.

Raise Mass:
Increasing mass is universally beneficial in reducing susceptibility to aerodynamic instability and motions. Similar to stiffness, the role of mass may sometimes be altered depending on whether loads or accelerations are important. For instance, increasing mass can reduce accelerations caused by longitudinal turbulence but not loads. On the other hand, increasing mass is always beneficial for reducing loads and accelerations due to vortex shedding.

5.3 Supplementary Damping
Increasing the damping is one of the most effective ways of reducing wind-induced loads and accelerations. Several techniques have been successfully used on other existing structures, including tuned mass dampers, viscous (oil) dampers, visco-elastic dampers and tuned liquid column dampers. It should be pointed out that increase in damping capacity is always beneficial for reducing loads and accelerations irrespective of the type of loading or phenomenon causing it.

6. CONCLUDING REMARKS

Wind tunnel tests account for building geometry, local climate and surrounding details and this leads to cost-effective and accurate wind loading on cladding and structural frames of tall buildings. In general, Code analytical methods are helpful for preliminary design and for simple situations, but provide conservative wind loads in most cases; underestimating in others. Presently, wind tunnel model studies offer the best estimate of the wind loading acting on a building for cladding as well as structural frame design. In general, cost of a wind tunnel study is dwarfed by savings in cladding and structural costs, which often add up to crores of rupees. In addition to the potential cost savings and accurate results, wind tunnel studies confirm that the architect's vision can be safely built and elevate litigation protection. Last, but not the least, most of the Codes themselves recommend wind tunnel testing for complex and tall buildings.

Codes themselves specify their limitations and encouraging more advanced/wind tunnel studies in case of complexity. Are we paying any attention to this in India? With our limited experience in India, in general, this portion of the code being neglected most of the time and this leads to the question of code enforcement. Most importantly, negligence on this part of the code will result in either too conservative design or unsafe design which we don't know. Please note that other developed nations as well as developing nations (for example China) are already in the race for such advanced studies. However, inertia still holds most of our designers from entering into innovative and challenging designs using wind tunnels. We hope the designers will understand pitfalls of codal provisions and take appropriate measures to carry out wind tunnel studies whenever it is necessary.

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STRUCTURAL ENGINEER – A SCAPEGOAT ?

VASANT S. KELKAR

INTRODUCTION

A Structural Consultant has to give certificates of safety/stability of a structure to several agencies like B.M.C., M.I.D.C., M.M.R.D.A., Factory Inspector, Lift Inspector. He has to also give Structural Audit Reports to building owners like Co-operative Societies and others. Some agencies like B.M.C., M.I.D.C. want the certificate in a particular format which can put a lot of responsibility on the Structural Consultant which is beyond the scope of his services and far too much compared to the remuneration he is paid.

The authorities want such certificates on their records so that in case of any mishap they can shrug off their own responsibilities and point a finger at the Structural Consultant who will become an unwilling Scapegoat.

The Chartered Accountants have a strong Institute. If you look at the audit reports they give of any limited company, you will notice that the Notes or Annexure to their audit have to be first signed by the Directors of the Company regarding their correctness before the C.A. signs them. Thus, a C.A. asks the Directors to confirm in writing many of the assumptions made by the C.A. in the audit.

It is necessary that the Structural Consultants also review the certificates they give keeping in mind the legal responsibilities and liabilities they will have to bear by giving such certificates. Organizations like ISSE and PEATA should hold a seminar to discuss these issues and take them up with the various authorities.

We shall discuss below formats of certificates given to some of the authorities.

2. MUNICIPAL CORPORATION OF GREATER MUMBAI (MCGM/BMC)

On completion of a building the Structural Consultant has to give a "Structural Stability Certificate" a part of which reads :

"I hereby certify that the Structural work of the above proposal has been carried out as per my structural design and details and that the structure is safe and stable for the purpose for which it is intended".

A Structural Consultant's scope generally includes only periodic supervision at site. So how can he certify that structural work has been carried out as per his designs and drawings? Even if he were doing full time supervision still he will not be able to certify this since he does not have a 24 hour control on the site.

MCGM has created a category of "Registered Site Supervisor" who has to be appointed by the owner for full time supervision at site for each project. Hence, Structural Consultant should first ask for a certificate from this agency that the work has been carried out as per his drawings. Like the Chartered Accountants even the contractor and/or the owners should be asked to certify that the work is carried out as per Structural Consultants drawings – since even the Registered Site Supervisor does not have a 24 hour presence at site and a lot of mischief can take place in his absence. The above sentence in the stability certificate therefore, could be reframed as follows:

"Based on the Structural designs and drawings given by me and my **periodic** supervision at site and on the basis of the **attached certificates** issued by the Registered Site Supervisor and the contractor/owner, I hereby certify that, **to the best of my knowledge and belief,** the structural work of the above proposal has been carried out as per my structural design and details and that …………"

Although the requirement of Registered Site Supervisor was made by B.M.C. (after its initiation and follow up by consultants like late Shri R. L. Nene) with good intentions, it is apparently not serving the intended purpose. Most of the builders ask one of their staff engineers (with a registration) to issue such certificate. This staff member may not be even supervising on the specific site. Thus, the supervision is not done by an independent agency who can point out faults of the builder/contractor. It is necessary that B.M.C. should insist on supervision done and certified by a Registered Supervisor who is an independent agency – as done by Project Management Consultants on major projects.

Certificate to be give to BMC at the start of a project, besides stating that the structure has been designed for seismic zone III as per IS 1893 etc. etc. also includes the following statement.

"All external walls shall not be less that 230 mm in Brick Masonry or 150 mm in Autoclaved Cellular Concrete block excluding plaster thickness".

The Structural Consultant is not generally concerned with specifications of nonloadbearing walls and their thicknesses except for taking their weights in calculations. The walls are specified by the Architect in his drawings and he should be asked to certify this clause. It is unfair to ask the Structural Consultant to certify for the walls. He does not even get paid any fees on the cost of nonstructural walls – the architect does. Specification of nonstructural walls could be changed to something not in accordance with this clause by architects after the structural work is complete i.e. after the structural consultant is almost out of site, without Structural Consultant's knowledge. But he will still be responsible for such changes. If the Structural Consultant wants to add this sentence in his certificate then the least he can do is to get a certificate stating this clause from the Architect and/or owner/builder and reframe this clause as :

"On the basis of the certificate issued by architect and/or owner/builder attached herewith, I certify that all external walls ……………."

Such demands may appear to be minor to some Structural Consultants but it is felt that there is no reason why a Structural Consultant should gullibly accept more and more legal liabilities for works which do not fall in his scope of services.

Few Clauses from typical audit report of any company given by a Chartered Accountant are given below for reference:

a) We have obtained all the information and explanations which **to the best of our knowledge and belief** were necessary for the purpose of our audit.

b) In our opinion, proper Books of Accounts as required by law have been kept by the Company, **so far as it appears** from our examination of such books.

c) On the basis of the **written representations received from the directors and taken on record by the Board of Directors,** we report that none of the Directors is disqualified as on 31st March, ……. from being appointed as a Director in terms of Clause (g) of Sub-Section (1) of Section 274 of the Companies Act, 1956.

d) In **our opinion and to the best of our information and according** to the explanations given to us, the said Accounts give a true and fair view in conformity with the accounting principles generally accepted in India.

3. GEOTECHNICAL INVESTIGATIONS

In old days the Structural Consultant visited site to inspect trial pits taken at site and assessed SBC of soil by visual inspection of exposed soil and by the resistance it offered when a worker drove a steel rod into it. This crude method of assessing SBC and founding level was ok when the buildings were small and sophisticated methods of soil investigations and expert Geotechnical consultants were not commonly available.

Now, for all projects the clients get borehole data taken on site on the basis of which recommendations for the type of foundation to be adopted, SBC or pile capacities to be used etc. are recommended by a Geotechnical consultant. The Structural consultant designs the foundation system on the basis of such report. It is natural to expect that the Geotechnical consultant will be responsible for the recommendations and professional opinions given by him and not the Structural consultant who is not an expert in the specialized geotechnical field.

However, if any problems such as due to uneven settlements etc arise due to incorrect recommendations in a Geotec report, still the BMC will hold only the Structural consultant responsible and liable for the problems since he is the only consultant issuing the stability certificate. Ideally, a separate certificate for the foundation system recommendations should be given by the Geotechnical consultant. Until BMC agrees to this requirement, it is necessary that in giving the stability certificate the Structural consultant at least adds a sentence like "The foundation design, SBC, Pile founding levels, their capacities are based on the report given by the Geotechnical Consultants M/s ……. ".

ISSE could formulate a proper draft of stability certificate considering the various suggestions above and any other suggestions received and circulate it amongst its members.

3. M. I. D. C.

The format of certificate to be given for a building constructed in M.I.D.C. includes the following statements :

"The work has been completed to my best satisfaction, the workmanship and all materials (type and grade) have been used in accordance with general and detailed specifications. No provisions of the act or the Building Bye-laws, no requisitions made, conditions prescribed or orders issued there under have been transgressed in the course of the work …… The building is fit for occupancy for which it has been erected/reerected or altered, constructed and enlarged …. "

The first sentence above mentions "work". It does not even say structural work so the Structural Consultant is liable for any problems arising out of even a faulty window falling and injuring someone. Secondly, if the Structural Consultant's scope only includes periodic supervision then how can he issue a certificate as above? Will any client pay him additional fees to do full time supervision?

Before start of construction M. I. D.C. requires that the Structural Engineer submit a "Form for Supervision" which is as follows :

"I hereby certify that the development work of plot and building for ……….. in plot no. ………… situated in ……….. block in M.I.D.C , T.T.C. Industrial area shall be carried out under my supervision and I certify that all the material (Type Grade) and the workmanship of the work shall be generally in accordance with the general specifications submitted along with and that the work shall be carried out according to sanctioned plan. I shall be responsible for execution of work in all respects".

That seems like a lot of liability the Structural Engineer has to bear. Again, the form states only "work" which can include anything besides structural work. Also, if the scope of the engineer is only periodic supervision how can he guarantee and certify that he "shall be responsible for execution of work **in all respects"**?

If you add words like "based on my periodic supervision", "to the best of my knowledge and belief" etc. then MIDC will not accept the certificate. So many Structural Consultants gullibly issue a certificate in the MIDC format thereby accepting a big liability on themselves. A dialogue with MIDC authorities by ISSE/PEATA is therefore necessary to make the format of the Certificates more rational.

5. AUDIT REPORTS GIVEN TO FACTORY INSPECTOR AND BUILDING OWNERS LIKE CO-OPERATIVE SOCIETIES

Several years ago there was a major collapse of a factory building in Taloja. Main reason was found to be overloading of building floors by the owners due to additions of several new machines. After this failure the Factory Inspector started asking for a Stability Certificate by a Structural Consultant to be given every five years.

Similarly, after collapse of several older buildings in Mumbai, BMC/Government has made it mandatory for the building owners to take Stability Certificate for their building from a Structural Consultant after every three or five years (depending on the age of the building).

Again, here, we see the passing of the buck to the Structural Engineer who can be blamed if any mishap should happen thereby relieving the Factory Inspector, BMC or the Government Department from the responsibility.

In a Structural Audit, a Structural Engineer can assess the condition of the building/factory mainly by visual observations. He may recommend some nondestructive testing to be done but usually co-operative Societies may not be keen to bear the additional costs. In visual inspection, areas covered by false ceilings, fixed furnitures, marble, granite etc. cannot be observed. So any distress in such areas cannot be assessed. In one of the factories, a steel column was encased in concrete. Outwardly it looked O.K. but when the concrete was removed for some alteration work it was seen that the steel column inside was completely corroded with its web almost nonexistent etc. due to effects of chemicals in the factory. It is difficult for the Structural Engineer to bring out in his audit report such deficiencies in the structure. Structural Audit, as done presently, is therefore, to put it in mathematical terms – "a necessary but not a sufficient" measure for assessing the safety of an old structure. This point should be brought to the attention of the authorities, building owners, co-operative societies and others who tend to think that the audit is "be all and end all" of required investigations for assessing and ensuring the safety of their building.

The Structural Consultant should add notes to Audit Report wherein he enumerates assumption made in giving the Audit Report. This can be:

- "No alterations in the structure of the building have been made without the written permission of the Structural Engineer since the last audit".

- "No new machinery, loads etc. were added in the factory building since the last audit" etc.

The owner of the building/factory should be asked to sign these Notes just as the Chartered Accountants ask Company Directors to do. ISSE members can suggest additional points to be added in such notes.

6. CERTIFICATE TO INSPECTOR OF LIFTS

One of the clauses in a typical certificate to be given to Inspector of Lifts by a Structural Consultant are as follows :

"This is to certify that all preventive measures have been taken to avoid any leakage from the overhead water tank into the machine room. The slab of the tank is made water proof".

Structural Consultant designs the water tank as per IS Code requirements. Specifications for internal waterproofing, if any, are decided by architect/owner. Hence, such a general clause as above about "preventive measures" should be also certified by architect/owners.

Another clause is as follows :

"This is to certify that the walls enclosing the lift well are constructed as per the requirements and have fire resistances of not less than two hours".

If the lift walls are in RCC, still a Structural Consultant cannot guarantee that they "…….. are constructed as per the requirements ……..." – he is not supervising construction for 24 hours. This should be certified by Registered Site Supervisor and contractor. Structural Consultant can give a certificate stating "…. are designed as per the requirements ………". If the lift walls are in brick or concrete blocks then their thickness etc. is specified by the architect and hence such certificate should be given by them. Structural Consultant can only certify the design of beams and columns surrounding the walls.

7. PROOF CHECKING OF DESIGNS

It is said that some consultants do not necessarily analyze/design medium height buildings for wind/earthquake loadings but just add some percentage in vertical loads (for column design) to account for wind/earthquake effects. Such practices can lead to unsafe designs. Hence, some control on designs done by the consultants is desirable. Presently BMC does not require structural designs/drawings to be proof checked. Only for buildings above 30 storeys, they have to be submitted to the Hirise committee. But it seems that this committee only does a general review of design basis but no detailed check of designs or drawings.

Agencies like MMRDA generally appoint another consultant to proof check designs of bridges, skywalks etc. submitted by a structural consultant. It is necessary that BMC also adopts such practice for proof checking structural designs/drawings of buildings – not just above 30 storeys but of lesser heights also. After all in Ahmedabad the buildings which collapsed during the earthquake were of 4 to 20 storeys in height.

8. CONCLUSIONS

Many Government agencies and municipalities ask certificates from Structural Consultants regarding safety/stability of a structure. The standard formats of these certificates show that they are not property drafted after taking into consideration the actual scope of work and the fees which are covered in the appointment of a Structural Consultant. Since any change in the format of the certificate will not be accepted by the department, many structural consultants gullibly give such certificates thereby taking on themselves huge additional responsibilities and liabilities should any mishap take place. It is necessary that ISSE and PEATA propose rationalized formats of certificates required by various authorities and ask them to change their formats to be more rational and realistic.

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EFFECT OF TEMPERATURE OF FRESH CONCRETE

Prof. R.G. Limaye & Devendra Limaye

1. INTRODUCTION

During analysis of strength results for concrete from some new constructions, it was noticed that strength in structural members was much lower than that specified even when the cube results were mostly satisfactory. Possible factors namely compaction and curing affecting results of strength were discussed in an earlier paper (2009). The effect of aggregates, wet or dry, was also discussed in an earlier paper (2007). In view of some resent observations it is thought appropriate to discuss the problems connected with the temperature of concrete in some details in the present paper.

2. FACTORS CONTRIBUTING TO THE TEMPERATURE OF FRESH CONCRETE

Concrete being a composite material each of the ingredients contributes to the temperature of concrete at the time of mixing.

a) Aggregates, when stored dry in open air can have relatively higher temperature above the ambient condition. Wet aggregates may have comparatively lower temperature. Chilled aggregates can be used to bring down the temperature.

Since the bulk of concrete, 70 – 75 % of the volume consists of aggregates, it's temperature predominantly governs the temperature of fresh concrete.

b) Water – Stored in tanks exposed to sun may have somewhat higher temperature. Cold water can be used to reduce the temperature of fresh concrete. Ice can be used in place of water to substantially reduce the temperature of fresh concrete, due to the effect of latent heat of melting.

c) Cement - As used in the mix may be at the ambient temperature or near about to start with. If directly exposed to sun during storage temperature may be somewhat higher. The type of cement, OPC or PPC, may not affect the result.

d) Fillers / additives – Like fly ash etc. may have similar behavior as that of cement.

e) Admixtures – being in very small proportions may not have any significant effect on the temperature.

When the ingredients are mixed together they will try to attain uniform temperature, the weighted average of the individuals.

3. EFFECT OF HYDRATION REACTION

As soon as water comes in contact with cement, hydration reaction starts at the surface of cement particles and lot of heat is librated being an exothermic reaction (heat of hydration of cement). Initially the rate of reaction and corresponding rate of heat liberation is very high for few minuets, and then reduces considerably. The amount of heat generated in the short period, depends on the chemical composition and the fineness of cement. Cement with higher proportions of tricalcium aluminate and tricalcium silicate would have higher heat of hydration and also higher rate of reaction and as such higher rate of heat liberation.

The dormant period following the initial peak is due to the formation of a film of hydration product on the surface of cement particles which breaks the contact between cement and water. Later through diffusion process and possible breaking of the film, the contact between cement and water is established again and reaction picks up. This time most probably corresponds to the initial setting time of cement.
The actual increase in the temperature of concrete due to this effect depends on the type of cement A/C ratio, W/C ratio and can be upto 20 – 25^{o}C above the temperature of concrete based on that of the ingredients.

4. TEMPERATURE OF FRESH CONCRETE AT THE TIME OF PLACING

The combined effect of the temperature of the ingredients and the initial heat of hydration determines the temperature of freshly mixed concrete. If this is exposed to sun it continuously gets heated further. This effect depends on the time of the day increasing from morning, reaching a peak around 2-4 pm and than decreasing.

Whether it is site mixed concrete or RMC this problem of temperature is similar in nature. In case of RMC and pumping, there is hardly any human interaction with this concrete. In case of normal working, the workers used to complain about the product getting hotter, if the temperature was higher than usually expected. With increased mechanization this feedback is practically absent. As a result of all these factors the temperature of concrete at the time of placing may be about 25^{o}C higher than the ambient condition. About 30^{o}C being the normal day temperature in a place like Mumbai almost all round the year, the temperature of concrete at the time of placing may very well be above 50^{o}C.

5. EFFECT OF TEMPERATURE OF FRESH CONCRETE AT THE TIME OF CASTING

All materials expand with temperature each having it's own thermal coefficient of expansion. So the volume of concrete at the time of setting is larger. To get an idea about the volumes involved it may be noted that the volume of water increases approximately 1% for every 20^{o}C raise from 4^{o}C where its density is maximum. When the concrete eventually cools down (in the night the temperature may be around 20^{o}C in Mumbai). It may also be noted that the surface temperature tends to be higher during the day and lower during the night compared with the temperature of the surroundings. During such cooling the concrete contracts and since it is already sets the change in length of the order of 0.04% can be accommodated only by way of cracks in the concrete. These may be wide spread microcracks or few relatively large cracks, as the tensile strain capacity of concrete at this stage is much lower. The effect of plastic shrinkage is not considered here.

Not more than 50% of total water is consumed in the hydration process for full hydration of cement. The free water remaining in concrete also undergoes these volume changes because of temperature, adding to the voids in concrete.

In some cases cracks are noticed in RCC slabs which are visible to the naked eye that run through the entire thickness of the slab. This can be clearly visible in the form of leakage of water. These temperature cracks are different in nature from the type of structural cracks due to loads. The orientation and location of structural cracks can be reasonably estimated from the structural details. The temperature cracks can not be easily estimated in this manner.

The structure of hydration products of cement also depend on the temperature of reaction, the net effect being to reduce the strength to some extent and make them more brittle in nature due to reaction at higher temperature.

The microcracks and voids formed in concrete due to this temperature effect tend to reduce the strength of hardened concrete. This is in addition to the effect of inadequate compaction and insufficient curing discussed earlier.

6. METHOD FOR REDUCING THE ADVERSE EFFECT OF HIGHER TEMPERATURE OF FRESH CONCRETE

There is special IS Code (IS-7861-Part 1- 1975) for hot weather concreting applicable beyond 40^{o}C. Various guidelines are listed for controlling the adverse effects. It is really advisable to implement these guidelines when the temperature of concrete is above 40^{o}C. The main emphasis is on controlling the temperature of ingredients of concrete including use of ice where necessary and start water curing as early as possible.

It is not intended to elaborate all these precautions here, it may be a topic for a separate paper.

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CRITICAL STUDY OF STRUCTURAL STEEL CONNECTIONS IN PRE-ENGINEERED METAL BUILDINGS

A. V. Patil & M. D.Vaishya

Abstract

In Pre-Engineered metal buildings connections plays a very important role in transfer of forces from one point to another. The present article discusses the End plate moment connections, Shear connections & Pinned Base Plate connection used in Pre-Engineered Metal Buildings. The various controlling limits on which the designs of above connection are depending are also discussed in the article.

A) Introduction:

Steel sections are manufactured and shipped to some standard lengths, as governed by rolling, transportation and handling restrictions. However, most of the steel structural members used in structures have to span great lengths and enclose large three-dimensional spaces. Hence connections are necessary to make such structures from one and two-dimensional elements and also to bring about stability of structures under different loads. Thus, connections are essential to create an integral steel structure using discrete linear and two-dimensional (plate) elements.

A structure is only as strong as its weakest link. Unless properly designed, the connections joining the members may be weaker than the members being joined. However, it is desirable to avoid connection failure before member failure for the reason that usually connection failure is not as ductile as that of steel member failure. Hence it is desirable to avoid connection failure before the member failure.

Usually cost of connections constitutes a major part of the total cost of steel structures. Hence, designers of connections have a great responsibility in reducing the overall cost of steel structures.

True behavior of connections is complex, variable and very difficult to analyze exactly. However, the connection design should be simple and straightforward, based on a clear understanding of the load transfer path, the effect of stiffness of elements in the path on the force distributed to the elements in the connection and the effect of ductility on the connection behavior. The detailing of connection should be simple and be based on repetitive use of standard practices to facilitate ease of fabrication and erection, thus assure speed and economy to the project.

B) Types of Connections in Pre-Engineered Metal Buildings:

In Pre-Engineered Metal Buildings the connections mainly used to connect built-up members are

1. End Plate Moment Connections

Moment connections transfer the moment carried by the flanges of the supported beam to the supporting member. Moment connections are assumed to have little or no relative rotation between the supporting member and the supported members. A Fully Restrained connection assumes the measured angles between intersecting members are maintained (i.e. no relative rotation) and there is full transfer of the moments.

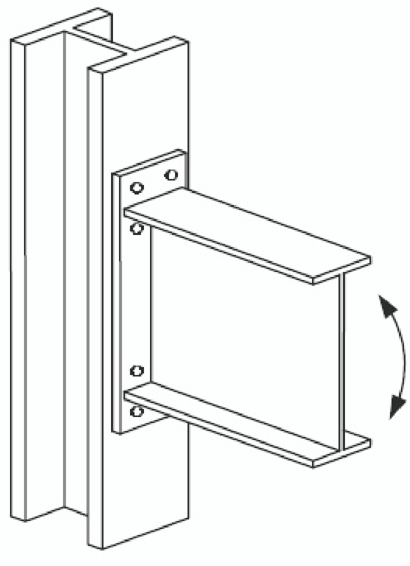
To transfer the tension and compression forces carried by the flanges, continuity between the supported beam flanges and the supporting member must be realized. Hence, the flanges of the supported member are attached to either a connection element or directly to the supporting member.

Transverse stiffeners are plates fabricated to fit between the flanges of the column at the point(s) of concentrated loading (tension or compression). Web doubler plates are steel plates that are fabricated to increase the overall thickness of the web of a section. Both types of components (transverse stiffeners and web doubler plates) are welded to the section to enhance the stiffness. The use of these components will increase fabrication costs, so it may be more economical to select a heavier column section or one with higher yield strength.

Extended end-plates are similar in appearance and orientation to shear end-plates. The primary physical difference is that the plate is longer than the depth of the supported beam as it must be attached to both the web and the flanges of the supported beam. The plate is usually fillet welded to the flanges and web of the supported beam; however complete or partial-joint-penetration welds may be used if the fillet size is excessively large. The plate is then bolted with high strength bolts to the supporting member.

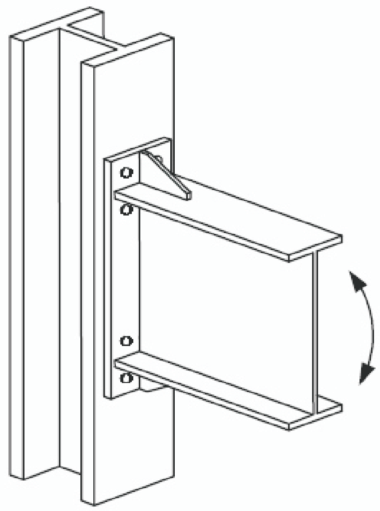
End-plate connections are classified based on the number of bolts used at the tension flange, such as

a) Four-bolt unstiffened



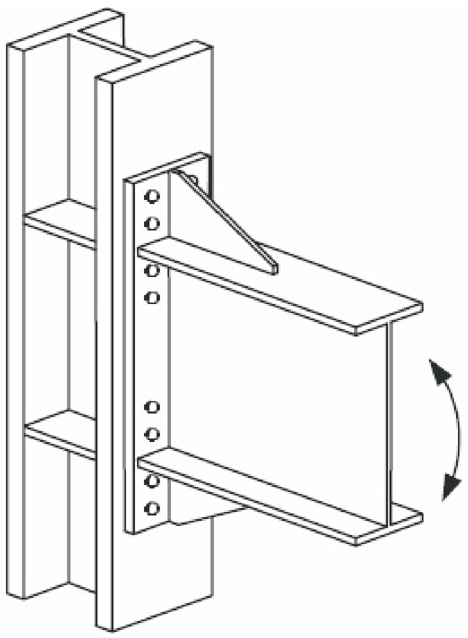
(a) Four Bolt Unstiffened, 4E

b) Four-bolt stiffened



(b) Four Bolt Stiffened, 4ES

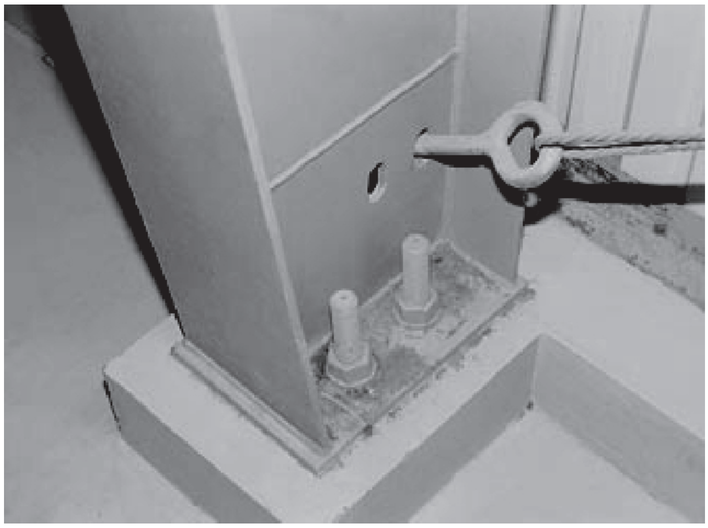
c) Eight-bolt stiffened



(c) Eight Bolt Stiffened, 8ES

The bolts in tension should be arranged in a symmetrical pattern with half above and half below the tension flange. At least two bolts should be used at the compression flange; these bolts serve primarily to carry shear forces. Furthermore, the bolts at the compression flange should be placed between the flanges of the supported beam whenever possible to reduce the required plate length. Extra bolts may be placed in the plate, near the neutral axis of the beam to ensure proper fit-up with the column and assist the compression flange bolts in shear transfer. Like their shear counterparts, extended end-plate connections require close accommodation of mill, fabrication, and erection tolerances. The beam may be fabricated short to accommodate field tolerances with shims furnished to fill any resulting gaps. The transverse stiffener is also part of the connection.

2. Pinned Column Base Plate Connections



Column base plates are used to provide a sufficient bearing area on the material below in order that the forces in a column are properly transferred to the foundation. Base plates usually anchor columns to a concrete foundation by anchor rods. The base plates of bracing struts may be bolted or welded to other steel members. The base plate can be attached to the column either by direct welding of the column to the plate or additional elements (i.e. angles) can be connected to the column that facilitates attachment of the base plate.

When the column is in compression, it bears directly on the material below. The compression load determines the size of the base plate. The actual connection of the base plate is effectively passive when there is only an axial compression load. If, however, the column is loaded in tension and/or shear, then the base plate connection becomes active. The base plate size is a function of the compression load and the connection to the base plate is a function of the shear and/or tension loads.

3. Shear Connection

Simple shear connections are assumed to have little or no rotational resistance. They are assumed to carry only the shear component of the load and are idealized as pins or rollers for design. Therefore, no moment forces are assumed transmitted by the connection from the supported member to the supporting member. The attachment of a shear connection may be made to the web of the supported beam, usually with the flanges unconnected.

Experimentally it has been shown that shear connections possess some amount of rotational restraint. For design purposes, ignoring this resistance produces a conservative result. The majority of the rotational flexibility of most

shear connections is achieved in the deformation of the connection element (plate, angle, tee, etc.) or through slotted or oversized holes. The deformation of the fasteners, if it is a bolted connection, or the welds, if it is a welded connection usually adds little to the overall connection flexibility.

The goal for shear connections is to have both adequate strength and sufficient rotational ductility. Shear connection elements are typically designed using thin and/or mild yield strength materials to provide rotational flexibility in excess of what the supported member requires.

Many shear connection elements can be either bolted on the supported side and welded on the supporting side, or welded on the supported side and bolted on the supporting side, or all-bolted or all-welded.

C) Limits for connection design:

Structural design is based on the concept that all structural members are designed for an appropriate level of strength and stiffness. Strength relates to safety and is essentially the capacity of a structure or member to carry a service or ultimate design load. Stiffness is typically associated with serviceability. Serviceability is concerned with various performance criteria of a structure or member during service loading and unloading.

Each strength limit state has a particular failure path across, through or along the element or member. The failure path is the line along which the material yields or ruptures. Serviceability limit states typically involve providing an appropriate amount of stiffness or ductility in a structural element. The serviceability requirements depend on the intended function of the member or element under consideration.

A connection may have many or only a few limit states. The controlling limit state can be either strength related or based on serviceability criteria. The controlling strength limit state is the specific condition that has the lowest resistance to the given design load. Initially, most designers tend to proportion elements based on strength requirements then check that the particular design meets applicable serviceability limit states, refining if necessary. The inverse design procedure is also acceptable: design for serviceability and then check strength. Regardless of the methodology the controlling limit state dictates the optimal design.

The following pages have descriptions and figures that explain the general applicability of the more common connection limit states. The applicability of any given limit state is dependent upon the specific connection geometry and loading. These figures are only a guide and are not meant to represent any and all possible combinations of limit states.

1. Block Shear Rupture

Block shear rupture is a limit state in which the failure path includes an area subject to shear and an area subject to tension. This limit state is so named because the associated failure path tears out a “block” of material. Block shear can occur in plies that are bolted or in plies that are welded. The only difference between the treatments of either the bolted or welded block shear limit state is that in the absence of bolt holes, the gross areas are equal to the net areas.

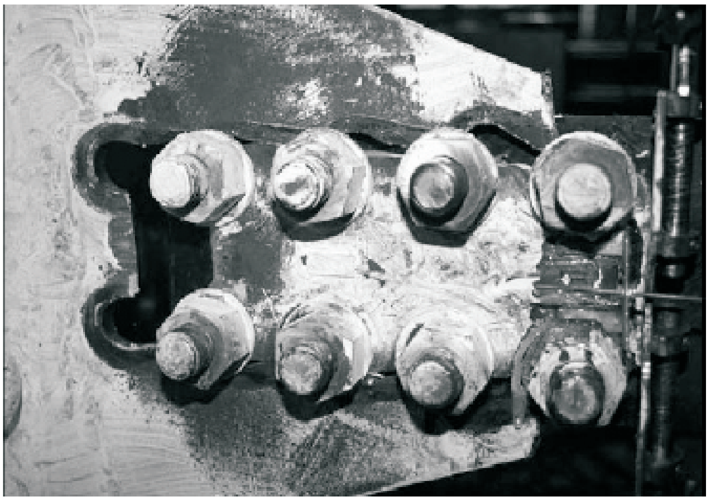


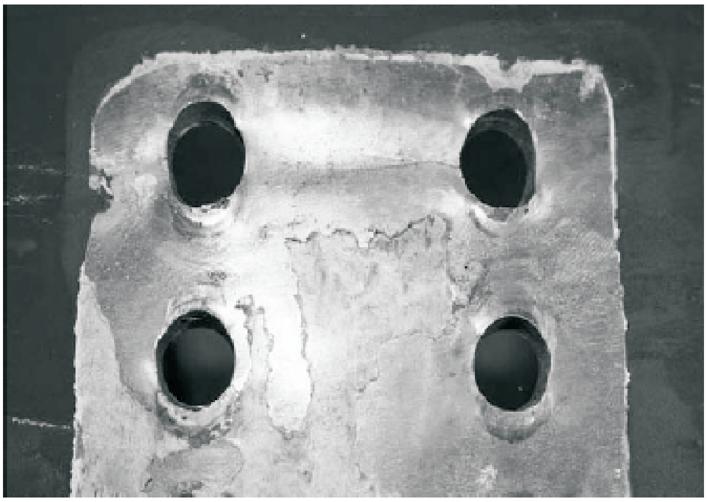
Figure shows the condition of the gusset plate well after the block shear rupture limit state has occurred

2. Bolt Bearing

Bolt bearing is concerned with the deformation of material at the loaded edge of the bolt holes. Bearing capacity of the connection is influenced by the proximity of the bolt to the loaded edge. Bolt bearing is applicable to each bolted ply of a connection.

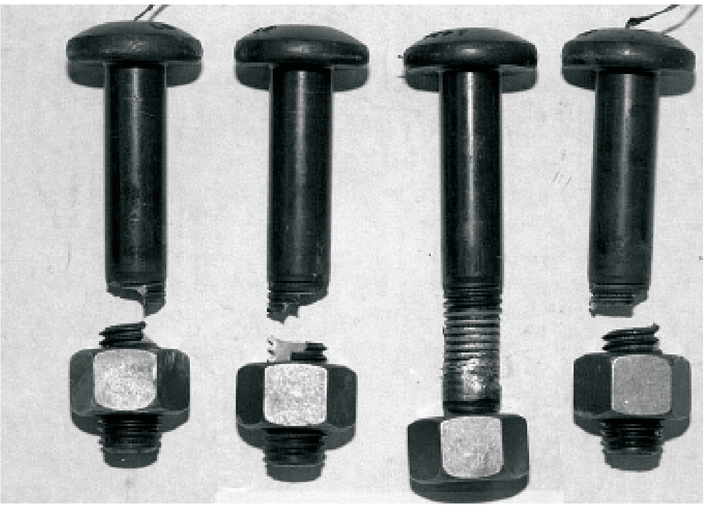
3. Bolt Shear

Bolt shear is applicable to each bolted ply of a connection that is subjected to shear. The shear strength of a bolt is directly proportional to the number of interfaces (shear planes) between the plies within the grip of the bolt that a single shear force is transmitted through. Single shear occurs when the individual shear force is transmitted through bolts that have two plies within the grip of the bolt. Additional plies further distribute the shear force. Three plies of material represent two shear planes, thus the bolt or bolt group is in double shear and has effectively twice the strength as single shear. It is important to realize that double shear, triple shear, etc. requires an individual shear force vector evenly distributed across the plies. There may be a condition where there are indeed two or more shear planes, but the forces are not evenly distributed. (e.g. double-sided connections)



4. Bolt Tension Fracture

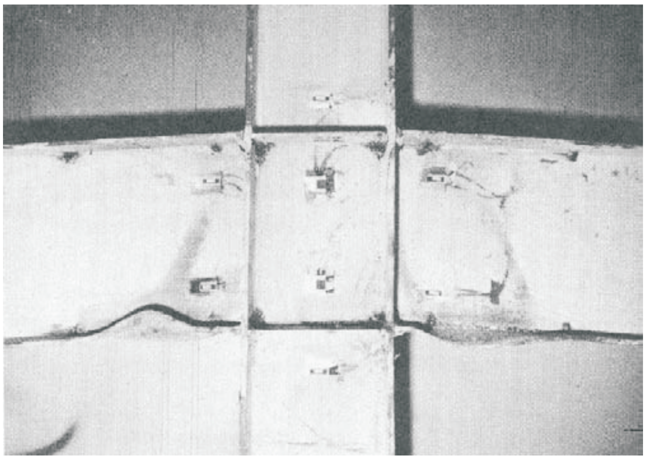
If bolts are subject to loading along their length then the bolt is loaded in tension. Bolts that fail in tension will do so within the threaded portion of the bolt, through one of the roots of the threads. This coincides with the least cross-sectional area.



5. Concentrated Force (Flange Local Bending, Web Compression Buckling, Web Crippling & Web Local Buckling)

Sometimes forces that are transferred from one member to another create localized deformation (yielding) or buckling. The applicable limit states depend on the specific connection geometry. The limit states for concentrated forces most often occur in seated connections and moment connections. For example, when the supported beam is coped, (i.e. flange material has been removed) the remaining web may be susceptible to web local buckling.

Since most moment connections provide continuity between the supporting and supported members, the flanges of the supported member transfer concentrated tension and compression forces to the supporting member. Flange local bending, web local yielding, web crippling and web compression buckling limit states must be investigated.



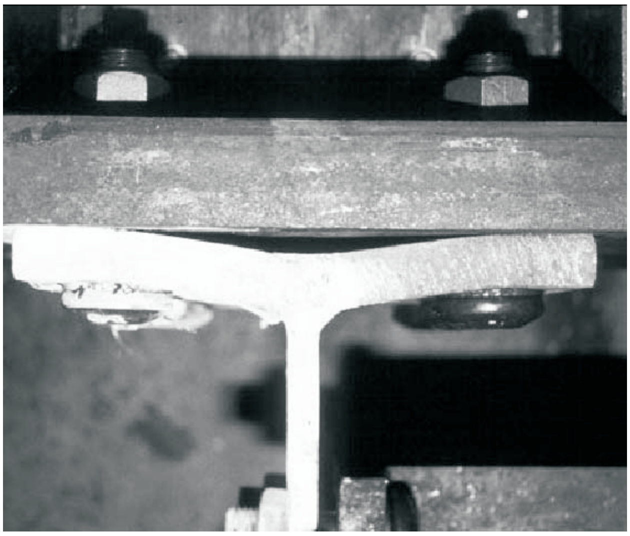
Flange Local Bending

Web Crippling Limit State

Web Local Buckling

6. Prying Action

Prying action is a phenomenon in which additional tension forces are induced in the bolts due to deformation of the connection near the bolt. Flexibility of the connected parts within the grip of the bolts creates these additional tension forces.



Prying Action

7. Shear Yielding & Shear Rupture

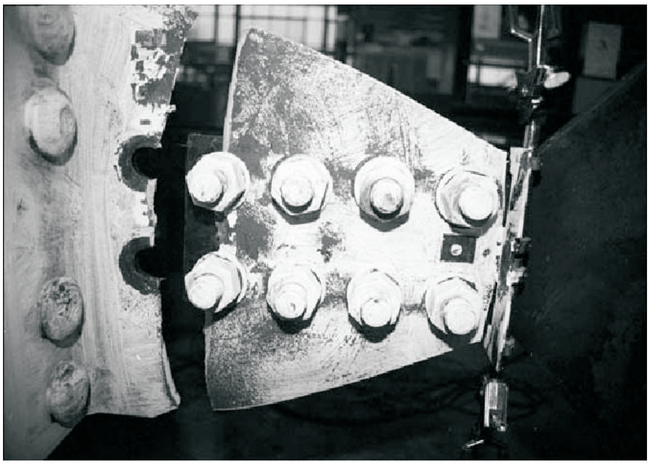
Most connections are subjected to the shear component of loading. Even moment connections must have provisions for shear transfer. Thus, those elements in the connection that are subject to shear forces must be investigated for shear yielding and shear rupture. Both limit states will apply regardless of fastening method (bolt or weld). For welded plies, without bolt holes, shear yielding will usually control over shear rupture. (The net area of welded plies without bolt holes is equal to the gross area. If the ratio of yield strength to ultimate tensile strength is less than 1.2, then shear rupture will generally control).

Shear yielding is a ductile limit state; it is a function of the gross shear area of the element. The failure path associated with shear yielding is linear in the direction of load from the top edge of the element to the bottom edge and through the thickness of the ply.

Shear rupture is an ultimate limit state; it is a function of the net shear area of the element. The failure path associated with shear rupture is also linear, in the direction of load from the top edge of the element to the bottom edge and through the thickness of the ply. If both flanges of the supported member are coped, then a potential shear failure path on the beam is present and shear yielding and shear rupture must be investigated for this member.

8. Tension Yielding & Tension Rupture

The tension yielding limit state is a function of the gross cross-sectional area of the member subjected to tension load. The tension rupture mode is a limit state that is a function of the effective net area. The net area is the reduced gross area due to bolt holes or notches. This net area is further reduced to account for the effects of shear lag. Shear lag occurs when the tension force is not evenly distributed through the cross sectional area of a member. Certain geometric areas of a section may have higher localized stresses. Shear lag often occurs in angle members when they are used as struts. The fastening (bolting or welding) is generally made along only one leg of the angle. When the angle is loaded in tension the leg that is fastened has a disproportionate share of the tension load. This unbalance causes a shear force to lag across the section.



Tension Fracture

9. Weld Shear

Weld shear is applicable to each welded ply of a connection. The failure mode for fillet welds is always assumed to be a shear failure on the effective throat of the weld. In a similar fashion as bolt shear, if the load path does not pass through the center of gravity of a weld group, then the load is considered eccentric. Eccentrically loaded weld groups are subject to a moment that tends to induce either additional shear (for in-plane loads) or combined shear and tension (for out-of-plane loads).

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5. AISC Design Guide Series – 4, Extended End Plate Moment Connections, 2nd Edition.

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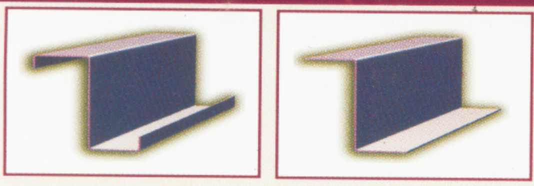

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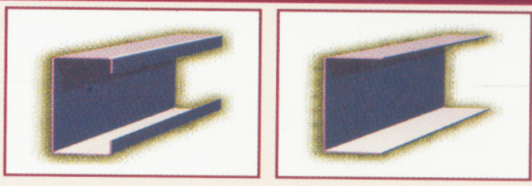

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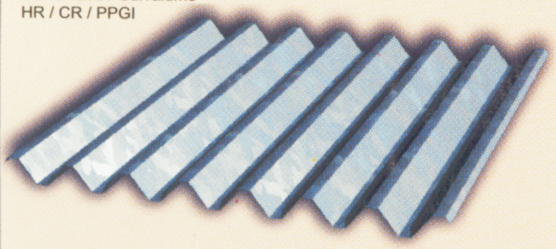

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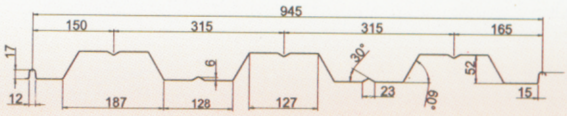

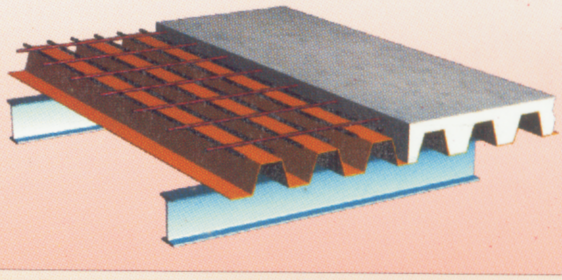

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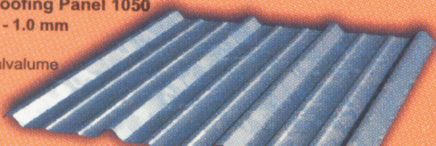

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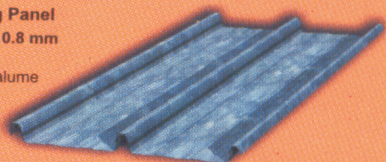

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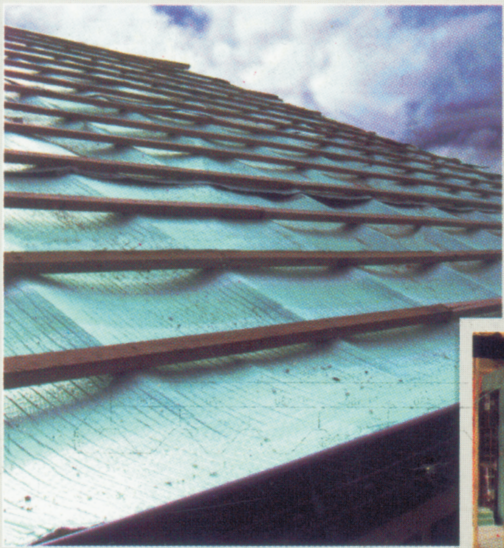

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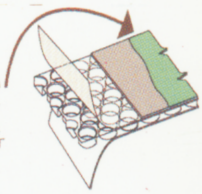

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