42 LEARNING FROM FAILURES: CASE STUDIES

1.0 INTRODUCTION

In pre-industrial societies, once a craft-based technique or thumb-rule for design was judged adequate for building an artefact, it was not considered necessary to develop it any further. The methods of design of buildings in those societies changed very slowly over time. Nevertheless, medieval society was indeed developing although at a relatively slow pace, leading eventually to the construction and erection of large and visible structures. Generally, these buildings symbolised the greatness or valour of a particular emperor or the glory of a particular God or religion. The impressive temples built by the great Chola or Pandia Kings in South India or the great Gothic Churches and Cathedrals in Europe (particularly in Italy) are excellent examples, which are impressive even by today's standards. The enhanced functional requirements of such buildings have continued to challenge the designers and technological pressures have continued to grow. For example, there has been an increasing demand to achieve the longest possible spans and the greatest possible heights in most prestigious buildings. In their desire to meet their clients' or patrons' needs, the designers did sometimes stray beyond the limits of contemporary technology and buildings and cathedrals collapsed as a consequence. This was the case with Beauvais Cathedral, which – when built - was considered to be the most daring achievement in Gothic Architecture. When its roof collapsed in *1284,* its restoration consisted of using tie rods of iron to hold the Gothic arches together, suggesting that the original designers had clearly over-reached themselves in the design of arches. (As is well known, arches are mainly compression structures, and develop horizontal thrust under purely vertical loads. We need sturdy supports to resist these thrusts. Clearly, there was design error in this case).

New developments in design are often the direct consequence of lessons learnt from previous failures, which are caused when the designers went too far beyond the state-of-the art or the contractors did not implement the design intent in the construction.

The development of scientific methods and reasoning, which started in the *17th* century, led to the ability to predict the forces to which a structure might be subjected. This led to the ability to validate structural designs – at least to some extent – in advance of construction. The process of industrialisation of societies also ensured the production of new materials whose properties could be predicted (unlike the natural materials - like stone - which they replaced). This combined with increase in knowledge and development of new materials actually led to the occurrence of more failures, principally as a result of enhanced demand for many types of novel structures for which there were no historical precedents, (for example, railway bridges).

2.0 THE NEED FOR FORENSIC STUDIES

Post mortem is an exact science. By employing it, we can establish the illness, which caused the death of the patient with a high degree of certainty. Many advances in Medical Sciences have been made possible by a systematic compilation of the results of post-mortem studies.

Engineering Designers, on the other hand, have been reluctant to reflect openly upon the causes of design failures, thus denying themselves and the profession an opportunity to understand the limitations of the particular design concept and improve the methodology. For example, by 1840 the British Engineers had simply abandoned the design development of suspension bridges, following the collapse of Menai Strait Bridge and suspension structures at Brighton Pier. All these failed in high winds, due to inadequate stiffening of the decks, a deficiency not recognised by the designers at the time.

Rather than interpreting the failures as an indictment of the form chosen, a contemporary American Engineer John Roebling collected case studies and established the forces - not hitherto considered - which must be designed against in order to build a successful suspension bridge. This resuscitated the suspension bridge technology. The famous English bridge-builder, Robert Stephenson, whose design of a Trussed Girder for Dee Bridge failed because of a very low factor of safety, was no doubt embarrassed but was candid enough to admit that **“ *nothing was so instructive to the younger members of the profession, as the records of the accidents in large works and the means employed in repairing the damage”*.** There were indeed plenty of bridge failures both in the U.S. and in Great Britain during the latter half of *19^th^* century and much discussion of the catastrophic failures did, in fact, take place. These influenced the design development of a number of new forms of the bridges. The cantilever bridge across the Firth of Forth (the Forth Bridge) designed by Benjamin Baker is a good example of this new development and was adopted by several bridge builders the world over. An editorial titled “ The Teaching of Failures” in Engineering News (*1887*) noted that ***“ …. There is no Engineer who, if he will look back upon the past and be honest with himself will not find that his most valuable and most effective instruction has come from his own failings…... Structures which fail are the only ones which are really instructive, for those which stand do not in themselves reveal whether they are well designed or so overly designed as to be wasteful of materials and resources…. The natural impulse of those who are in anyway responsible for failures….is to keep the matter as quiet as possible ….something not difficult to do in cases where there is no great catastrophe or loss of life”.***

It is clear that much can be learned through the failure of a structure rather than a study of structures, which are successful. The proper appreciation of the causes of failure helps us to refocus on our conceptual understanding of structural behaviour. We could then assess our analytical models, which are essential for successful design practice, and help us to exercise proper engineering judgement.

Many design decisions are inevitably based on engineering judgement, which does not merely come from an understanding of theory or a powerful command of computational tools. Even extensive design experience in an academic context can only provide limited perspectives in engineering judgement. Most fruitful lessons in engineering judgement are obtained from the case histories of failures, which point invariably to examples of bad judgement; these, naturally, provide guideposts for negotiating around the pitfalls in conceptual design. They also offer invaluable insights into the potential trip-wires in early attempts at innovative design and construction. In many cases, important new principles of engineering science may be brought out in the study of failure case studies.

Some structural failures are caused due to:

(1) Poor communication between the various design professionals involved, e.g. engineers involved in conceptual design and those involved in the supervision of execution of works.

(2) poor communication between the fabricators and erectors.

(3) Bad workmanship, which is often the result of failure to communicate the design decisions to the persons, involved in executing them.

(4) Compromises in professional ethics and failure to appreciate the responsibility of the profession to the community at large could also result in catastrophic failures.

Other common causes of structural failure are summarised below:

- lack of appropriate professional design and construction experience, especially when novel structures are needed.
- complexity of codes and specifications leading to misinterpretation and misapplication.
- unwarranted belief in calculations and in specified extreme loads and properties.
- inadequate preparation and review of contract and shop drawings.
- poor training of field inspectors.
- compressed design and/or construction time.

In this chapter some case studies of failure are presented. In each case study, the possible learning points, technical aspects and ethical implications are also discussed.

3.0 POOR CONCEPTUAL DESIGN

3.1 Tacoma Narrows Bridge

The destruction of the Tacoma Narrows Bridge by aerodynamic forces subsequently revolutionised the thinking of structural engineers, on how wind loading could affect large slender structures. This is a good example of errors in Conceptual Design.

In *1940*, Tacoma Narrows Bridge was opened across Tacoma Narrows in Washington State. On Nov *7, 1940*, with a wind speed of about *60 km/h* (well below the design wind speed), the bridge began twisting and oscillating violently. As a result the tie down cables intended to stiffen the bridge snapped, causing the entire structure to crash into the river below.

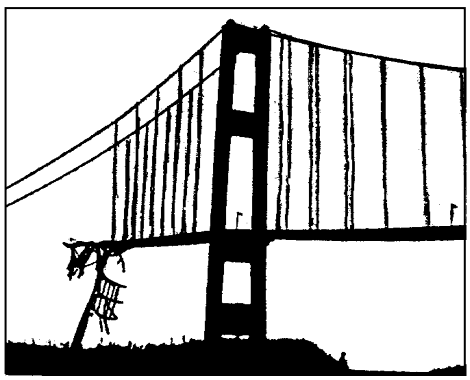


Fig. 1: Tacoma Narrows Bridge

Investigations (subsequent to the collapse) showed that the excessive vertical and torsional oscillations (which occurred prior to failure) were the result of extraordinary degree of flexibility of the structure and its relatively small capacity to absorb the dynamic forces. The deck was too narrow for the span and thus its torsional rigidity was inadequate. The plate girders, which were provided for stiffening, had insufficient flexural rigidity and little torsional rigidity. Their elevation caused wind vortices above and below the deck in moderate and steady winds. From the day bridge was opened very substantial horizontal and vertical movements of the deck in waveforms were noticeable even in moderate wind and high traffic.

The failure was indeed caused by a lack of proper understanding of aerodynamic forces and knowledge of torsional rigidity in the whole profession. It was not realised by the designers that the aerodynamic forces (which had proven disastrous in the past to much higher and shorter flexible suspension bridges) would affect a structure of such magnitude as the Tacoma Narrows Bridge, despite the fact that its flexibility was greatly in excess of that of any other long span suspension bridge.

It is clearly dangerous to exceed the design paradigm without fully understanding the forces one is dealing with and the limitations of applicability of current design concepts.

3.2 Millennium Bridge at London

This *320 m* span Aluminium and Stainless Steel Bridge across the River Thames in London was opened on *10* June *2000* amidst a lot of fanfare. It is the first river crossing to be built in London, after Tower Bridge (completed in *1884*) and links St. Paul's Cathedral (in the North Bank) and the new Tate Modern and Globe Theatre (in the South Bank).

In many ways it is an unusual structure. Sir Norman Foster, a famous British Architect claimed to have designed it in association with an Artist, Sir Antony Caro and the Engineers were Ove Arup and Partners, a distinguished firm of consulting Engineers. From the start, Foster emphasised the innovative nature of its design. The objective was " to push the suspension bridge technology as far as possible, to create a uniquely thin bridge profile, forming a slender blade across the River Thames". Jonathan Duffy, a BBC commentator remarked " It sounds great and on paper, probably looked sublime, but often reality is the harshest judge of cutting edge Architects". The bridge was made of Aluminium decking and stiffened by suspension cables in the horizontal plane. No attempt was made to stiffen it in the vertical plane.

During the first weekend (*10-11* June *2000*), some *160,000* persons crossed the bridge essentially because of its novelty. As people began to cross, it became apparent that the bridge was swaying several inches from side to side. The transient population on the bridge swayed drunkenly as they walked in synchrony, as if choreographed. The bridge was indeed wobbling dangerously over very deep waters. Many felt sea-sick while crossing. It was obvious that the bridge was not adequately stiffened to resist gravity loading. An American visitor remarked that " the design of the bridge looks as flimsy as some of the rope bridges seen in Indiana Jones films....".

The bridge had to be closed to traffic after having been open only for two days. The Engineers/Designers are hoping to install dampers (similar to shock absorbers) to reduce the oscillations to a minimum (acceptable) level.

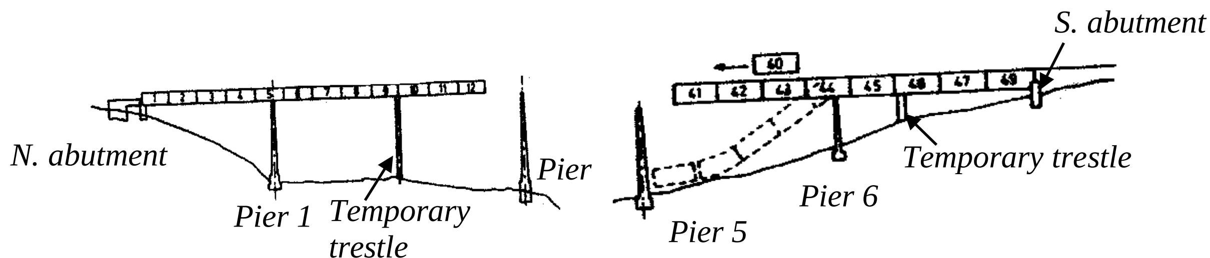
This case study illustrates the dangers of over confidence. The designers had extrapolated the established Technology into untested (and dangerous) situations. It is true that dozens (if not hundreds) of Bridges have been built all over the world. Nevertheless it remains the case that all the suspension bridges (as indeed all the structures) should be adequate both with respect to "<u>strength</u>" as well as "<u>stiffness</u>".

4.0 DESIGN INADEQUACY

Cleddau Bridge, Milford Haven, (UK)

The failure of three box girder bridges during erection in *1970* in quick succession revealed the need for a radical re-examination of the prevailing design methodology for Thin Plated Structures and their erection.

On *2* June *1970*, Cleddau Bridge in Milford Haven failed during its erection by cantilevering segments of the span, out from the piers. The bridge was designed as a single continuous box girder of welded steel. The span that collapsed was the second one on the south side. The boxes were fabricated in sections and moved over the previously built sections, aligned in place and welded. The collapse occurred when the last section of box for the second span was being moved out along the cantilever. This section slid forward down the cantilever buckled, at the support and collapsed into the river (Fig *2*), killing four men, including the site-engineer.

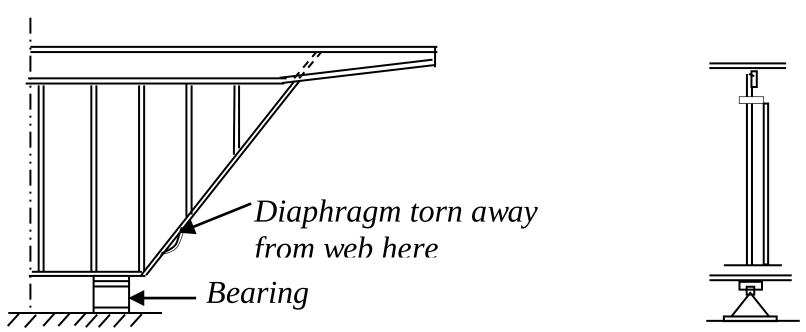


Cantilever on north side of bridge ***South side cantilever***

Fig. 2: Failure of Milford Haven Bridge

Investigation of collapse showed that the collapse was due to the buckling of the diaphragm at the support (i.e., at the root of the second span being erected). The diaphragm was torn away from the sloping web near the bottom. This caused reduction in the lever arm between flanges resisting negative bending moment at the support. The tendency of the bottom flange to buckle was inevitably increased by the reduction of the distance between the flanges, as this increased the force needed in each flange to carry the moment with the reduced lever arm.

The support diaphragm was, in effect, a transverse plate girder, which carried heavy loads from the webs of the plate girder at its extreme ends and was supported by the bearings as shown in Fig *3*. It was therefore subjected to a hogging bending moment and a large vertical shear force. The shear of the transverse girder and diffusion of the point load from the bearings were compounded with the effects of inclination of the webs of the main bridge girder. These produced an additional horizontal compression and out-of-plane bending effects caused by bearing eccentricity.



Section

Elevation

Fig. 3: Diaphragm over Pier6 of Milford Haven Bridge

The total load transmitted by the diaphragm to the bearings just before collapse was computed as *9700 kN*. This load would not have caused any problem provided the diaphragm was designed to carry it. Allowing for likely values of distortion and residual stress, the calculated design strength, using design rules that were drafted subsequently, was found to be as low as *5000 kN*. Thus, the failure was essentially due to design inadequacy.

5.0 POOR COMMUNICATION BETWEEN THE DESIGNER AND THE FABRICATOR

Hyatt Regency Walkway Collapses

The case study presented here focuses on the professional responsibilities of structural engineers as they assume overall responsibility for their designs. It also focuses on the need for a uniform understanding of the means by which specific responsibilities are communicated between the members of project team.

On *7th* July *1981*, a dance was being held in the lobby of the Hyatt Regency Hotel, Kansas City. As spectators gathered on suspended walkways above the dance floor, the support gave way and the upper walkway fell on the lower walkway, and the two fell onto the crowded dance floor, killing *114* people and injuring over *200*.

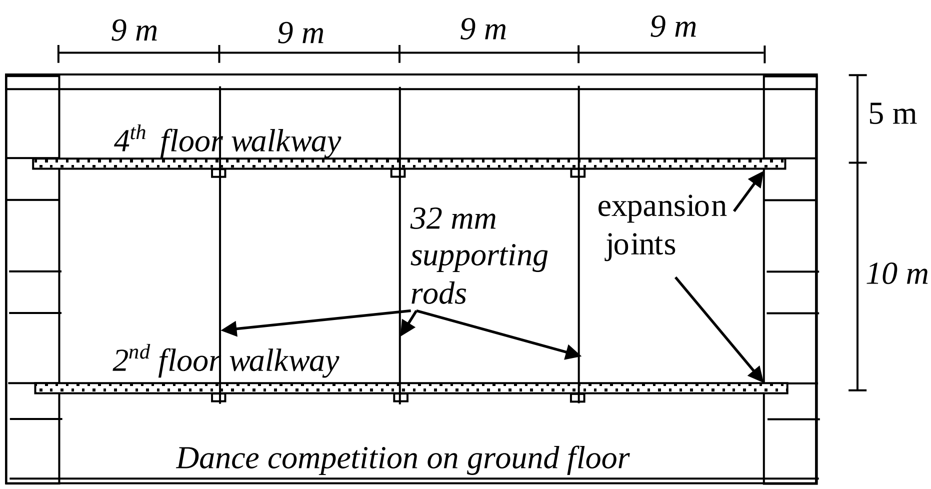


Fig. 4: Kansas City Hyatt Hotel: arrangement of walkways

The two walkways were supported above one another and suspended from the ceiling by hanger rods as shown in Fig *4*. The walkways were supported on box beams, which were made of two steel channels, welded together.

In the original design a single rod supported the two walkways as shown in Fig. *5(a)*. But the originally designed hanger detail for the two walkways was altered at the time of fabrication as shown in Fig. *5(b)*. The second floor walkway was suspended from the fourth one as shown. As a result, the connection between the fourth floor cross beam and the hanger supported double the load originally intended as shown in Fig *6(b).* Examination of the box beams supporting the upper walkway after the collapse showed that the upper hanger rod had pulled through the beam. The beam design was also unsatisfactory, and this condition was aggravated by the increased load on the nut. The nut pulled through the box beam as shown in Fig. 7.

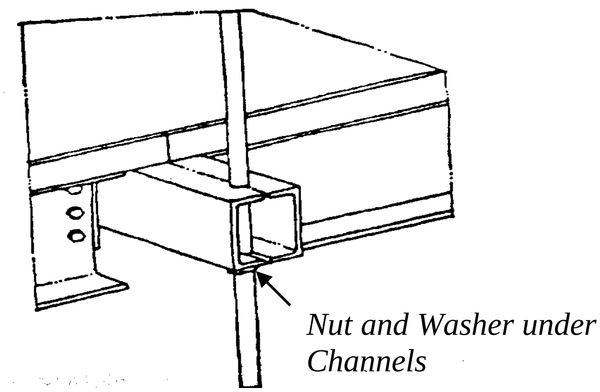


Fig. 5(a): Hyatt Regency Hanger Details As-Designed

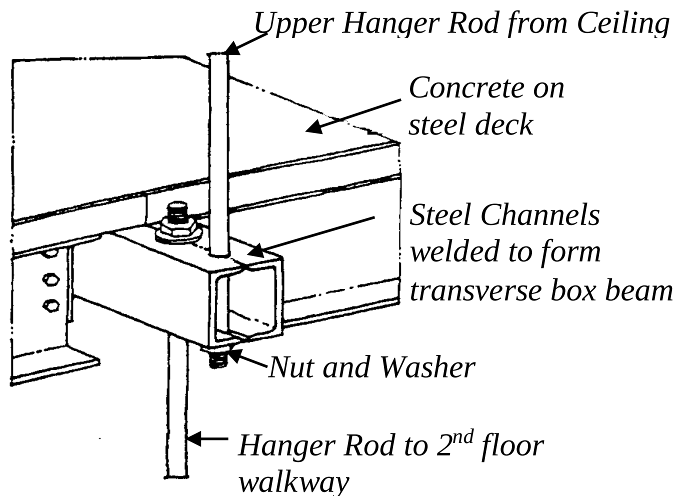


Fig. 5(b): Hyatt Regency Hanger Details As-Built

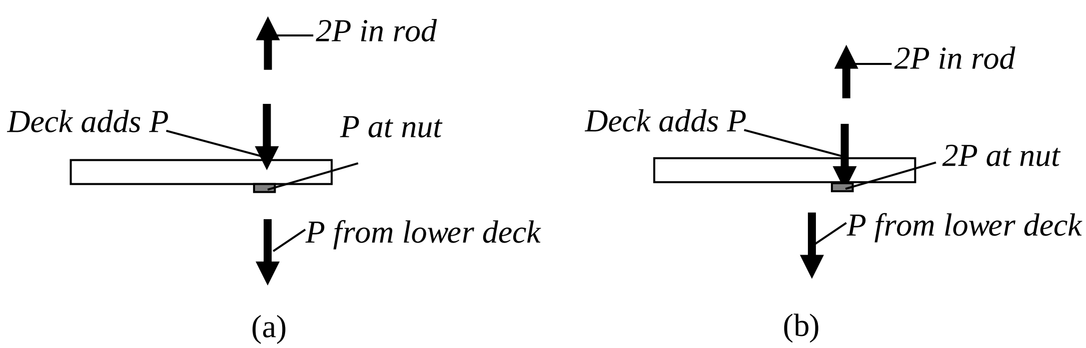


Fig. 6: Free-Body Diagram (a) As Designed (b) As Built

After the investigations, it was found that the steel fabricator who built the hanger detail requested a change in detail from the originally designed detail. The engineer approved it without checking the calculations. This accident occurred due to the carelessness of the engineer concerned as he failed to understand the importance of the details he had changed. It also illustrates the importance of understanding the force flow in the joints and that of what is often considered as minor detail.

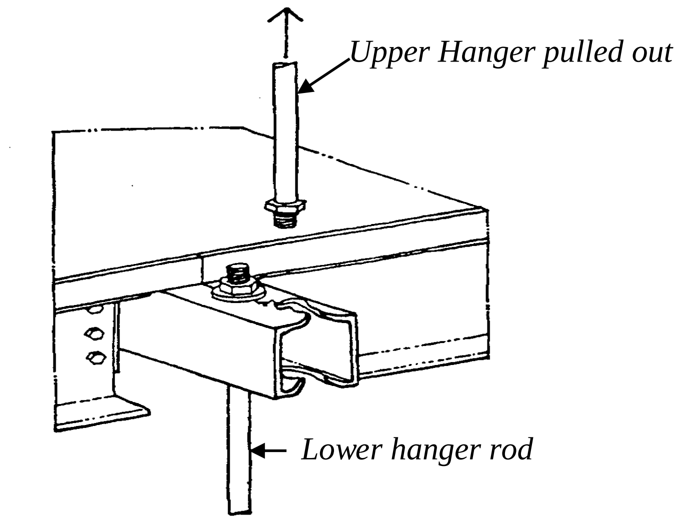


Fig. 7: Pulled -Out Rod at Fourth-Floor Box Beam

6.0 POOR DETAILING

King's Bridge, Melbourne

The next case study is an example of poor detailing compounded by poor communication and a lack of necessary inspection.

Kings Bridge in Melbourne is one of the relatively few examples of failure in service. It was opened in *1961*, but only *15* months later, on *10th* July *1962*, it failed when a *45*-ton vehicle was passing over it. Collapse was only prevented by a wall, which had been built to enclose the space under the affected span.

The superstructure consisted of many spans in which each carriageway was supported by four steel plate girders spanning *30 m*, and topped with a R.C.C deck slab as shown in Figs. *8(a)* and *8(b)*. Each plate girder bottom flange was supplemented by an additional cover plate in the region of high bending moment. The cover plate was attached to the flange by a continuous *5 mm*-fillet weld all around (see Fig. *9)*

An investigation into the possible cause of failure indicated that the failure was due to brittle fracture and many other spans of the bridge were in danger of similar failure. Cracks were found in the main tension flange plate of the affected span under seven of the eight transverse fillet welds. One crack had extended such that tension flange was completely severed, and the crack had extended halfway up the web.

Investigation also revealed that difficulties were experienced during welding. Special care to avoid unnecessary restraints during welding was not taken, despite the specifications. The longitudinal welds were made before the transverse welds. As a result there was a complete restraint against contraction, when transverse welds were made. Moreover, transverse welds were made in three stages. In some instances cracks were caused in the main flange plate by the first run and later covered up by a subsequent one. In many other cases a crack was caused by the last run and later covered up by priming paint before the girders left the factory. The penetration of later paint coats into the cracks showed that they had often extended further before the bridge was opened for traffic.

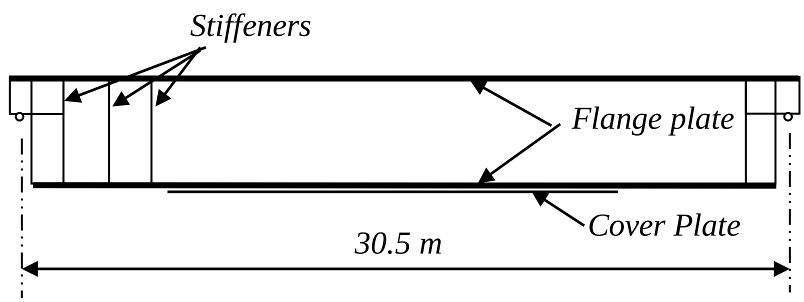


Fig. 8(a) Girder Elevation

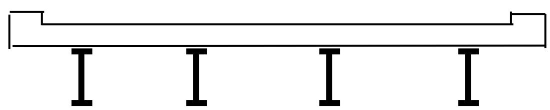


Fig. 8(b) Girder Cross section

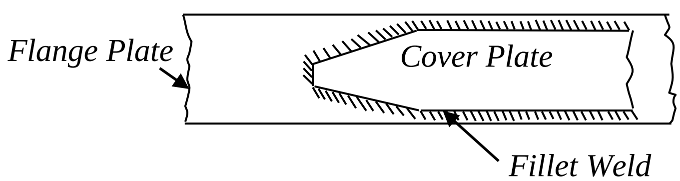


Fig. 9: fillet weld

The results of investigations clearly indicated that, the failure of King's Bridge was due to carelessness of those who fabricated the girders as well as those who inspected the bridge. It was also found that the most likely and most dangerous cracks were regularly missed by inspectors, who had carefully got the less harmful longitudinal cracks cut out and repaired.

7.0 POOR JUDGEMENT

Quebec Bridge Failure

The following case study is intended to show how, the errors of the judgement of the engineers could lead to the failure of the structure and loss of so many lives.

On *29* August *1907*, the partially constructed south cantilever arm of the Quebec Bridge in Canada collapsed killing *75* workmen due to the grave error made in assuming the dead load for the calculations. Even when this error was subsequently noticed the designer chose to ignore it, relying on the margin of safety inherent in his design.

The bridge was intended to carry rail traffic across the St. Lawrence River at Quebec. It was designed and built under the supervision of Theodore Cooper, doyen of American bridge builders in the late *19th* century. The bridge consisted of giant truss cantilevers on two main piers, with a suspended span in the middle.

Two compression chords (made of lattice construction) in the south cantilever arm failed by the shearing of their lattice rivets. As the distress spread through the entire superstructure, the nineteen thousand tons of the south anchor, the cantilever arms and the partially completed centre span thundered down onto banks of the St.Lawrence River and into the water the bridge had been designed to cross.

Investigation report on the major events leading to the accident is summarised below.

The bridge was put for a ‘ design and construct ‘ contract although the original specification of T. Cooper was followed. Originally the bridge was designed for a span of *1600 ft*. Later, the span was increased to *1800 ft*, considering both engineering and expenditure. For this Cooper had provided modified specifications that would allow for high unit stresses. Accordingly, the design calculations were revised but due to an oversight the added dead weight in the increased span was not included in the calculations and fabrication of steelwork began.

After placing the first steelwork on site, Cooper realised that the weights of the fabricated components were not corresponding to previously estimated dead loads and that the working stresses were in fact *7* to *10%* greater than that allowed by specification. But Cooper decided that the increase in stresses was safe and permitted work to continue.

During construction, Cooper was once informed that some problems were encountered in riveting the bottom chord splices of south anchor arm on account of their faced ends not matching. But Cooper instructed that the work should continue, as it was not a serious matter.

When work on the central, suspended span proceeded, the rapidly increasing stresses (and the consequent buckles) on the compression members became intolerable. Later the end details of the compression chords began to buckle. The buckles started developing in an alarming fashion leading to the collapse of the structure.

Thus the bridge, subject to hasty design decisions, came to an untimely end. The court of enquiry found a number of factors, which had contributed to the accident. Among these were the unusually high permissible stresses allowed in the specification and the lack of communication between consultant, designers and the site management. It was recognised, however, that these were factors, which only served to aggravate the main cause of failure, which was that the designer had failed to provide the main compressive load bearing members with adequate strength. In the subsequent enquiry and investigations it became clear that the lacing system and the splice joints of the compression members were not able to resist the effects of the buckling tendency of the compression members

8.0 POOR INSPECTION AND MAINTENANCE

Silver Bridge Collapse

Silver bridge collapse is considered to be one of the failures that had been very influential. It led to the approval of the *1968* National Bridge Inspection Standards by the U.S. Congress. Built to specifications, this American Suspension Bridge was completed in *1928* and failed in *1967*. The cause of failure was a fracture in an eyebar link resulting from a crack which had grown through stress, corrosion and corrosion fatigue.

A brief report on the causes of failure is summarised below.

On Dec *15, 1967* Silver Bridge, considered to be first eyebar suspension bridge in the United States, collapsed without warning into the Ohio River. The bridge was spanning Ohio River between Point Pleasant, West Virginia, and Gallipolis, Ohio. The collapse occurred when the bridge was crowded with heavy traffic resulting in the loss of *46* lives and nine injuries.

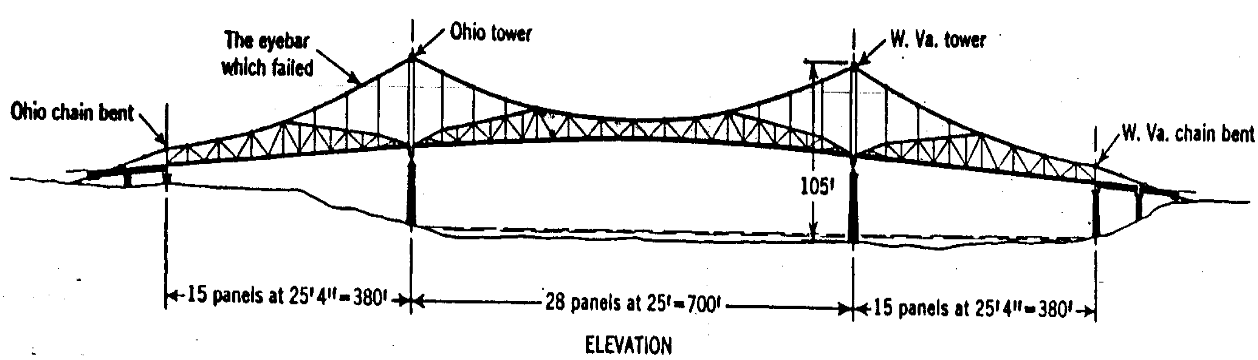


Fig. 10 a: Silver Bridge at Point Pleasant

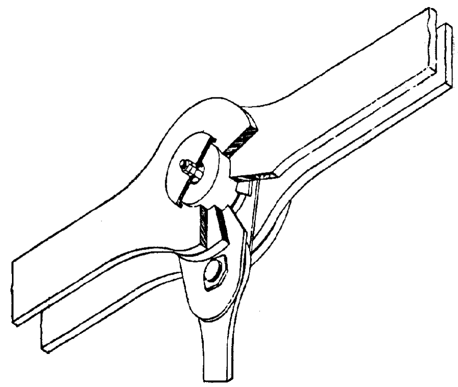


Fig. 10 b: Detail of Eyebar Chain Joint

A thorough investigation revealed that the collapse of the bridge was caused by the failure of the eyebar at the first panel point west of Ohio tower as shown in Fig.*10.*

At the beginning, the first joint of the eyebar, west of the Ohio tower came apart. As a result of the separation of the joint and the failure of the eyebar, the Ohio tower fell eastward. The collapse continued eastward, causing the West Virginia tower to fall eastward. Thus once the continuity of the suspension system was severed at first panel point west of Ohio tower, the unbalanced forces on each side of that joint caused the bridge to totally disintegrate.

Investigations showed that there were two main elements in the design and construction of the chain that caused the failure - extremely high tensile stresses and corrosion on the inside of the eyebar.

Moreover the chain was composed of two bars, which meant that the breaking of one bar would inevitably result in total instantaneous collapse of the entire bridge. It was also found that the factors of safety for the eyebar design were too low compared to the requirements of the original design. No consideration was given in the design to secondary stresses arising from

- inaccuracies in the manufacture of the bars.
- stresses created by unbalanced loads.
- unequal distribution of the total stress between the two eyebars.
- lack of complete free movement around the pins.

Another undesirable feature of the design was that the eye, where the pin fits, was elongated *3 mm* in a horizontal direction for ease of erection. This detail created an air space where corrosion could develop undetected and unabated. The inspection or lubrication of the inside of the head of the eyebar in the Silver Bridge was impossible without dismantling the joint

Thus the combination of high tensile stresses and corrosion caused a crack on the inside of the eyebar, under the pin at a location of a manufacturing flaw about *6 mm* in size.

The tragedy of the Silver Bridge did not go unnoticed and unrecognised. Its collapse created a huge uproar in the United States. The first major benefit was that it led to the approval of the *1968* National Bridge Inspection Standards by the U.S. Congress (Systematic Bridge Inspection and Evaluation). Another major benefit arising out of the Silver Bridge tragedy is the attention paid to eyebar trusses and details. In particular, tension members composed of two eyebars became suspect and required special attention. Such lower chords were strengthened or replaced. A third benefit was the attention given to all connections: floor beams to trusses, stringers to floor beams, trusses to bearings, and so on. It became necessary to inspect these details with great care.

9.0 POOR CONSTRUCTION

Cracking in suspended floors of a school building.

The next case study (*The Structural Engineer, 1994)* concerns the cracking of a slab caused by the constructor not paying attention to the requirements of the Serviceability Limit State. Although this example concerns a R.C.C slab, it is regarded as important for structural engineers involved in Steel Design as frequently his designs incorporate R.C slab as a component in Composite Construction.

In the school building reported herein, cracks were noticed in the suspended floors. All the cracks were found on the top surfaces of the one-way slabs, on each side of, and parallel to, the beams that were supporting them.

In the course of investigation, surface crack widths were measured. A covermeter survey was carried out near the cracks. In addition, the slab was pierced at a number of locations in order to supplement the covermeter survey and also to measure the slab and plaster thickness. The range of measurements and a comparison with the specifications in the structural drawings are shown in Table *1*.

Table 1 Suspended floor slab parameters

	Specification	Measured range
Slab thickness (mm)	100	83-106
Reinforcement spacing (top) (mm)	225	235-400
Effective depth of top steel (mm)	75	35-55

Table *1,* clearly indicates that poor workmanship was responsible for the cracking observed. There were adverse deviations from the specifications in slab thickness, effective depth, as well as reinforcement spacing.

10.0 POOR CONSTRUCTION PRACTICES

Roof Truss Collapse

The next case study is taken from the *Journal of Performance of Constructed Facilities, ASCE (1992)*. In this example the designer of record did not have any field inspection responsibilities. Construction was left in the hands of contractors who, whether experienced or not, used "customary" installation techniques that left the trusses inadequately braced.

The roof of a shopping centre (consisting of several timber roof trusses) collapsed after two days of snow and rain. Most of the trusses on one side of the centre beam had collapsed and the top of the load-bearing wall had been pushed out. The centre beam was undamaged and undeflected. Investigations showed that the building had been in service for six years. The structure was a rectangular building consisting of *3.7m* high concrete block bearing walls and a wood truss supported system. A steel beam supported by steel pipe columns was installed in the centre of the building running along the longer dimension. The truss system consisted of two monopitch trusses placed peak to peak forming a conventional "A" shaped roof. The pair of trusses spanning between the sidewall and the centre beam, rested on the top flange of the beam but weren't connected. The roof system acted as two independent halves. And the building was subdivided into several stores by non-load bearing partition walls. It was found that the trusses that were still standing on the affected side of the beam had no lateral bracings and none of the internal diagonals had any bracing. The lateral bracings were provided only for the vertical members of the trusses at the beam bearing. The first diagonal members in compression were found to be out of plane by several centimetres. They had failed as load bearing members. An analysis showed that these members, when unbraced, exceeded the allowable length to depth ratio for in plane compression. They were not able to withstand the requisite snow loading. Thus the trusses had been left unbraced and understrength at the completion of construction.

Shop drawings for the trusses produced by its manufacturers showed that two lateral members were required for the first diagonal and one brace was required for the second diagonal. This information was either never furnished to the installer or ignored by the installer. During investigation it was also revealed that though the manufacturers had developed handling and bracing recommendations, many truss installers ignored these guidelines.

11.0 HIGH ETHICAL STANDARDS AND TIMELY ACTION PREVENT A FAILURE

The fifty nine storey crisis

The next case study is an example of the high ethical standards and professionalism characteristic of a competent engineer involved in areas of safety and welfare of the public.

The Citicorp centre, a fifty nine-storey tower in Manhattan, New York designed by William J. Le Meassurier, would have faced a major disaster if a serious error in its design had not been detected in time. He acknowledged the errors done by his team, prepared new plans and got all the necessary changes put into effect to avert a possible disaster.

The Citicorp centre was the seventh tallest building in the world at that time. The tower had twenty five thousand individual steel jointed elements behind its aluminium skin. It was supported on four massive two hundred and seventy eight meter high columns, which were positioned at the centre of each side allowing the building corners to cantilever twenty two metres out. Its wind bracing system consisted of forty-eight braces (in six tiers of eight), arrayed like giant chevrons. A tuned mass damper was also provided to dampen the wind-induced vibrations (due to the heavy mass of the damper, the severity of the vibrations would be reduced).

The trouble started when Le Meassurier learned that the wind braces designed by his team were not checked for diagonal winds, which would result in a forty percent increase in strain in four out of the eight chevrons. Moreover despite the welded joints specified, bolted joints were provided by the contractor as the welded joints were considered to be expensive and stronger than necessary. But if the bracing system was sensitive to diagonal winds, so were the joints that held it together. The joints must be strong enough to resist the moment, which was the difference between the overturning moment caused by wind forces, and the resisting moment provided by the weight of the building.

At any given level of a building, the value of compression would remain constant. Even if the wind blows harder, the structure would not get heavier. Thus immense leverage could result from higher wind forces. In the Citicorp tower, the *40%* increase in stress produced by diagonal winds caused a hundred and sixty percent increase in stress on the bolts at some levels of the building. The assumption of *40%* increase in stress from diagonal winds was theoretically correct, but it would go higher in reality, when the storm lashed at the building. This fact was completely disregarded by his design team. The weakest joint was discovered at the thirtieth floor and if that one gave way, catastrophic failure of the whole structure would have resulted.

The statistical probability of occurrence of a storm was found to be one in every sixteen years. This was further reduced to one in fifty five years if the tuned mass damper (which had been installed) was taken into account. But this machine required electric current, which might fail as soon as a major storm hits.

Le Meassurier learnt of these design faults after the building was completed and handed over. Nevertheless, he acknowledged these errors because keeping silent would mean risking people's lives. So he brought these errors to the notice of the owners of the building and persuaded them to invest in his newly prepared rectification scheme. Since the bolted joints were readily accessible, the new proposal was to strengthen the joints, which were weak. All the weak joints were reinforced by welding heavy steel plates over them.

His honesty, courage, adherence to ethical and social responsibility during this ordeal remains a testimony to the high ideals of a true professional.

12.0 INDIAN EXPERIENCE

All the Case Studies reported so far in this chapter have been compiled from published reports and journals from UK and US.

The culture of reporting failures and the lessons learnt from them has not yet developed in India. In many cases the reluctance of the Engineers concerned is also due to the fear of potential legal action and resulting claims.

12.1 Improper design leads to heavy restoration

The next case study is an example of errors committed in design due to inexperience and wrong assumptions.

The case study concerns a factory building near Nellore, India. The building was of size *25.7 m X 52.5 m*. The roof was made up of steel Pratt trusses supported on concrete columns. The entire truss was exposed except for the bottom chord members, which were embedded in concrete slab.

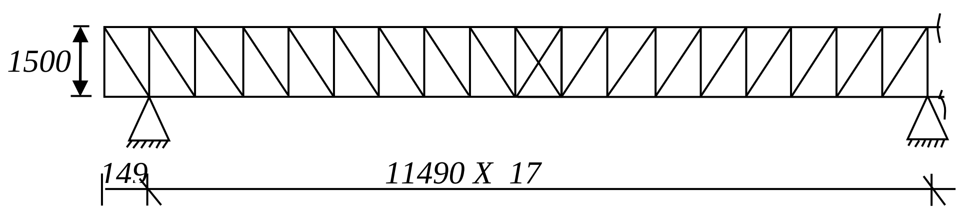


Fig. 11: Elevation of Truss

After curing the concrete slab, when the scaffoldings were removed, the deflection of the roof was found to be *100 mm (>L/325)*. In order to find out the cause of these disproportionate deflections, the truss was reanalysed. The configuration of the truss is shown in Fig. *11*. From the analysis, it was found that most of the members of the roof truss were not safe. After performing several analyses, it was concluded that in the original design, the designer might have miscalculated the loads. In India, industrial buildings were normally covered by asbestos cement sheetings. Hence the original designer of the truss, due to his inexperience, might have considered the truss to support Asbestos sheeting, instead of heavier concrete slab. The analysis considering only AC sheet roofing confirmed that all the members of the roof would be safe for the reduced loading.

By the time the investigation started, the expensive machinery, which were to be housed inside the building had arrived and were under installation at various parts of the building. Hence the repair of the roof had to be done without affecting the work of installing these machineries. Hence it was decided to strengthen the top and bottom flanges of the top chord members by welding extra plates on them. It was also decided to weld angles inside the web members of the truss. Since the bottom chord member was inside the concrete slab, it was not possible to add anything to the bottom chords.

It was also recommended to provide temporary supports to the truss at *1/3* point for welding extra plates and angles. But the contractor did not provide the temporary supports and welded the extra plates and angles. This resulted in the buckling of the web of the top chord members. The web members in one or two trusses had also buckled. Hence after careful consideration it was decided to reduce the span of the truss which would eventually reduce the forces in the members. After consultation with the manager of the plant, the locations of these intermediate columns were fixed and then the work was carried out.

12.2 Restoration of a factory building

The next case study deals with another design error made by the designer due to his overconfidence.

A factory building located at about *100 km* from Bombay collapsed during a windstorm in *1994*. The building was built using cold-formed channel members. The layout of the building is shown in Fig.*12* and the elevation in Fig.*13*.

The structure was provided with column bracings in every sixth bay. However no gable end bracings were provided. Extra columns were provided at the gable end to support the cladding. The structure was covered with asbestos roofing and all the sides were covered with asbestos sheet cladding. The structure was designed to support a *4t* gantry in each bay. A crane bracket supported by the column (see Fig.*13*) supported the gantry.

In order to understand the failure of the structure, the original design was examined. The main causes of failure were found to be,

(i) wind loads were not estimated properly as per *IS:875*.
(ii) column and rafter sections were found to be inadequate to resist the load; they did not even satisfy the main ℓ/r ratio specified in the Code.
(iii) during erection, the bracings were not connected properly to the main members.

For the restoration of the structure, the designer was asked to use the same sections and produce a design, which would not increase the cost of the project considerably. Hence same channels were used; but their spacing was altered to form a box section with diagonal bracings of channel section. The span and bay width were kept the same. This arrangement increased the moment of inertia of the section along the frame and perpendicular to the frame. The rigidity of the structure increased considerably in both the directions and the bending stress was found to be well within the allowable range of stresses.

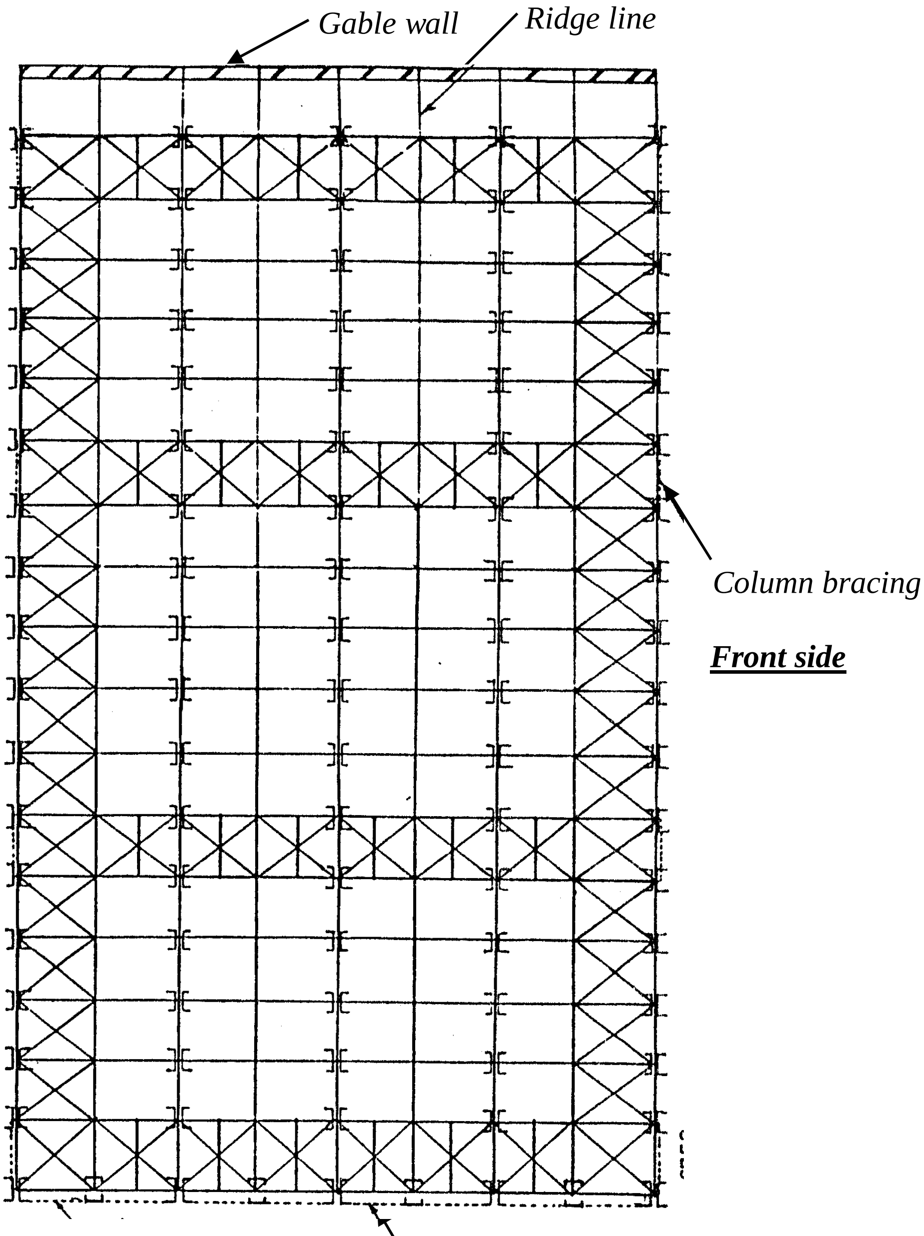


Fig. 12: Layout plan of the factory building

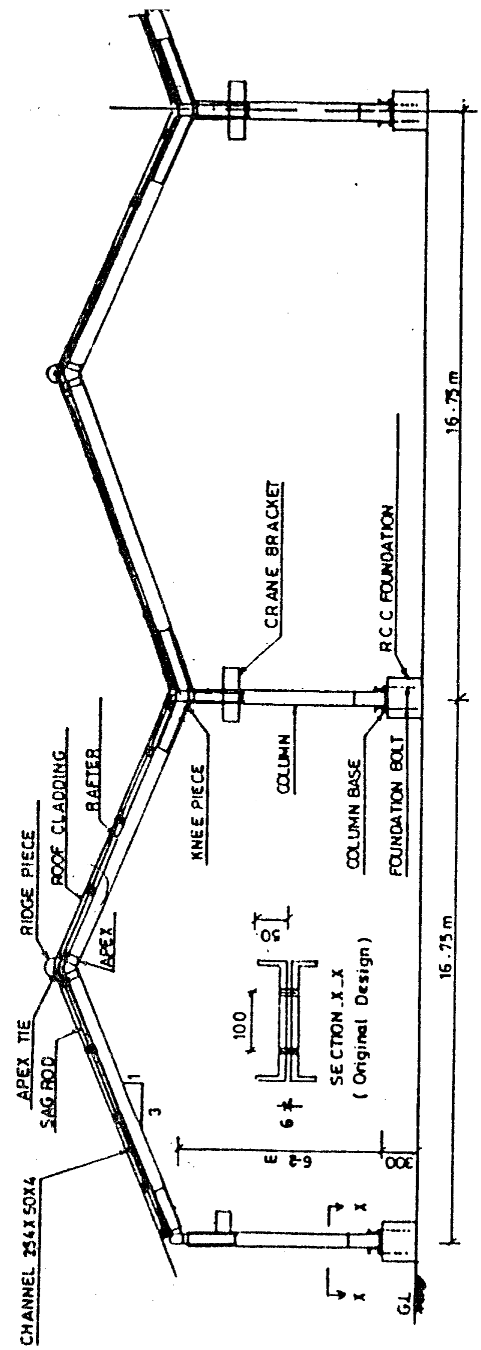


Fig. 13: Cross section of collapsed frame

12.3 Improper detailing results in delayed commissioning

In the next case study, wrong detailing adopted by a designer resulted in delayed commissioning of the project.

A *144 m X 60 m* factory building was constructed at Cochin, India. The plan is shown in Fig.*14*. The building was made up of portal frames spanning *60 m* and placed at *6 m* intervals. The portal frames were supported alternately on columns and on lattice girder that was placed longitudinally at the mid-span. The portals and columns were made up of four angles, which were laced to form a box section. The lattice girder and the central column were made up of *4* channels, laced to form a compound section. The sides of the building were also covered by cladding from *1.5 m* above G.L.

After the erection of portal frames and placing of asbestos sheets, some problems were encountered. The purlins and side cladding girts got twisted. This resulted in the cracking of some AC sheets. Some columns (especially those supporting the partitions) were not straight and gave a buckled column appearance. Most of the side cladding girts were sagging.

A careful investigation showed that the purlins were not detailed properly and were placed in the wrong orientation, which resulted in the torsion of the purlin sections. This is explained in Fig. *15*. Moreover the detailer had given a connection detail as shown in Fig.*16 (a)* to connect the purlins at rafter points as against the correct detail in Fig.*16 (b)*. Due to this the purlins got twisted till the tip of the purlins rested on the rafter section.

By this time all the machinery of the plant had arrived and the erection of these was in progress. It was also a costly proposition to remove all the AC sheets, correct the detailing errors by refabricating the joints and relaying the AC sheets. Hence after several rounds of discussions, it was decided to replace only the cracked AC sheets and to adopt a temporary solution as shown in Fig.*17* which would arrest further twisting of the purlin.

The sagging of the side cladding girts was due to the fact that the sag rods were not anchored by providing diagonal sag rods at the ends. This was rectified. The other problem was due to the fact that the fabricator was not experienced in cold rolled steel sections. Since these sections were flexible and made of thin sections, the fabricators simply bent the columns and fixed them at the required place. These mistakes were also rectified. However, these corrective measures delayed the starting of the plant production by about six months.

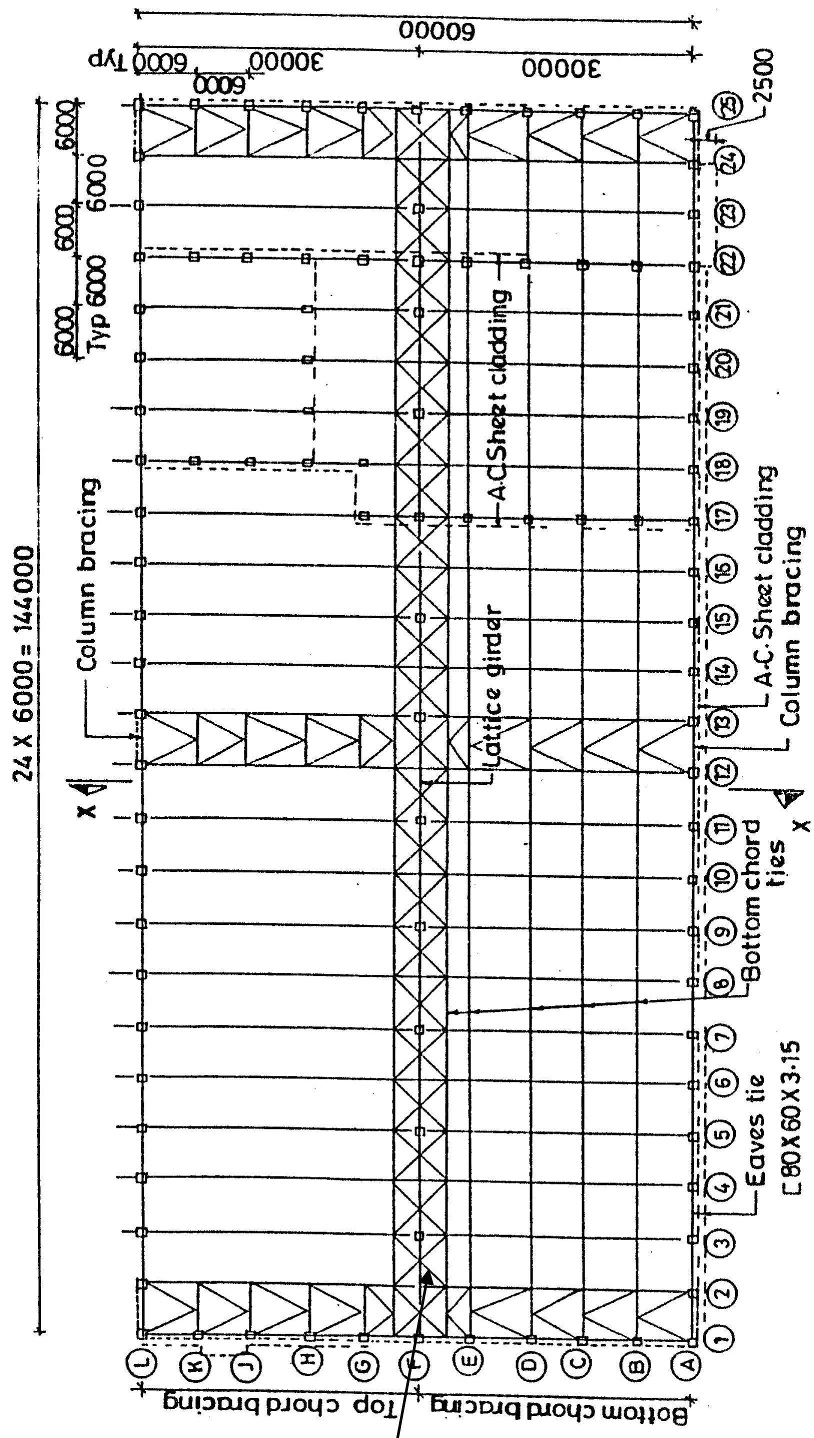


Fig.14: Plan view of the structure

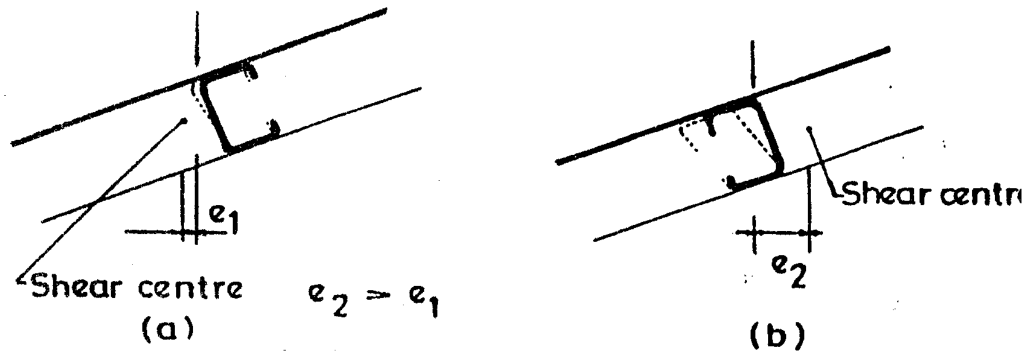
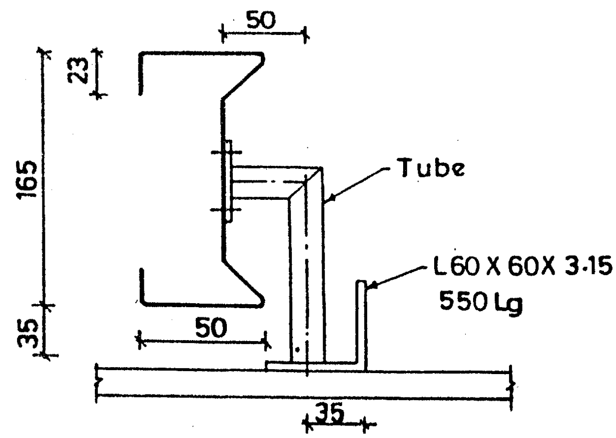
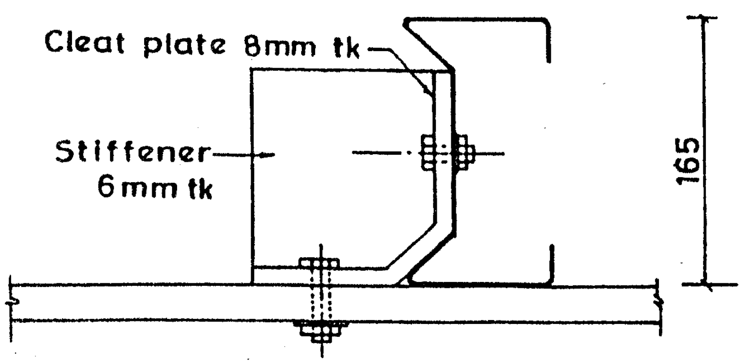


Fig. 15: Torsion of purlins



(a) Connection detail as adopted at site



(b) Correct connection detail

Fig.16: Adopted and correct seating detail of purlin

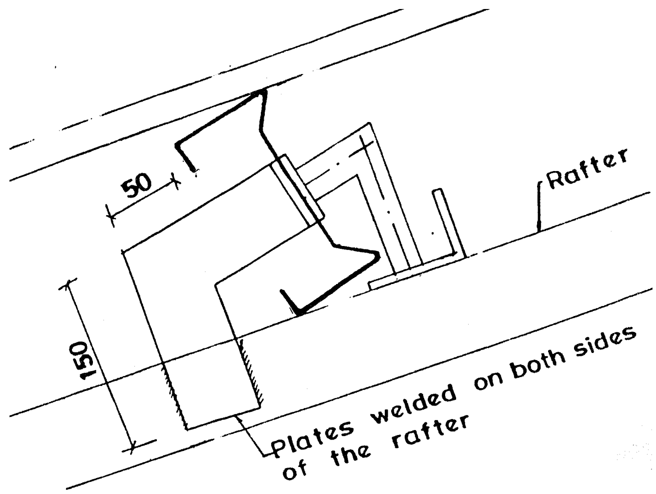


Fig.17: Solution adopted to hold the purlin in place

13.0 Lessons learnt from the Gujarat Earthquake of 26 January 2001

Over 30,000 people are reported to have died in the earthquake of magnitude 7.9 (on the Richter Scale), which hit parts of Northwest Gujarat on Republic Day 2001. Thousands of people - who had led respectable life-styles till then - have seen their life-savings vanish and their lives irrevocably destroyed. Do we have to accept this human suffering and carnage with fatalism and detachment? Or can we ***protect*** our buildings by careful designs and thereby save lives?

Earthquakes in California of even larger magnitudes have not resulted in losses in life of this magnitude, because the buildings in that State are required to comply with the State's Earthquake-resistant Design Codes. Indeed, when an earthquake of similar magnitude hit Seattle, U.S.A. on 1 March 2001 ***there was not even a single loss of life and only a few persons were injured, none seriously***. This is the result of the extensive retrofitting that was carried out in the city during the 1970's. ***The Central Public Works Department has claimed that none of their buildings in the Kutch area had suffered any damage during the earthquake due to their sound structural designing.*** Clearly, the technology exists to ***protect*** our buildings and ***prevent*** loss of lives and - equally clearly - the building designs in Gujarat have not been subjected to checks on their structural adequacy and on their safety by qualified Structural Engineers and Soil Engineers. (IS 13920 pertains to "Ductile Detailing of Reinforced Structures subjected to Seismic Forces" and IS4326, to "Earthquake resistant Design and Construction of Buildings".) Clearly, these Code-prescribed checks were not insisted on before the appropriate authorities approved the designs for construction. Thousands of these buildings collapsed subsequently, despite the availability of design guidance.

There is an irrational reluctance to use Steel Structures among many professionals in India largely due to misinformation, lack of confidence, or inexperience. Steel is inherently ductile; steel structural components, when stretched or elongated under overload, do not fail or collapse. On the other hand, concrete is a fracture-sensitive material, which cracks under tensile forces. As a material, concrete is inherently unsuitable to sustain overloads or repeated loads caused by earthquakes nevertheless reinforced concrete was used as the preferred material of choice in practically every building. The only steel-concrete composite multi-storeyed building under construction in Ahmedabad suffered no damage due to the earthquake.

The ignorance of currently available technology is compounded by the willingness of the Indian Builders and clients to accept shoddy and primitive construction, particularly in concrete structures. There is very little quality control of concretes used in Indian buildings. There are endemic problems such as low cement content, poor quality reinforcing-steel, inadequate concrete cover to reinforcing steel and non-existent site supervision; all of these will need to be remedied systematically to prevent repetition of this disaster.

The following is a partial list of inadequacies and infractions, identified by professional engineers who visited Ahmedabad after the disaster:

- Most buildings that had a planning approval for (Ground floor plus 4 levels) had a further floor added illegally; buildings with approval for (Ground floor plus 10 levels) had two further floors added illegally. Swimming pools and/or roof gardens (on soils spread to a depth of one metre over the roof) were added features of some of these luxury buildings. Obviously they were unsafe and triggered the collapse.

- Columns loaded from above were terminated at the free end of a cantilever at the second floor level. There was no provision for transferring these loads on to the foundations.

- Most buildings that collapsed were built on stilts, with the ground floor being used for car parking. The flexible columns at the ground floor level failed rapidly during the earthquake and initiated the progressive collapse of these buildings. This type of failure could have been prevented by concrete infill walls or suitably designed bracings to the ground floor columns.

- The falling concrete debris from collapsed structural components caused substantial loss of life during the earthquake. ***It is essential that the structural integrity of the building be maintained even if the individual members had failed***. Each building should be effectively tied together at each principal floor and roof level in both directions. Reinforcing bars in concrete floors should be effectively anchored to the beams at its edges, so that these floors would function as edge-supported membranes, rather than fall down on the floor below, thereby causing damage.

- In many buildings, only the lift shaft was saved, and the rest of the building collapsed around it, as the former was not effectively connected to the latter. Well-designed lift shafts would effectively function as core walls and provide the much needed stability in multi-storey buildings.

- Buildings that are unsymmetrical in plan will be subjected to unexpected twisting which would cause substantial damage. Re-entrant corners should be avoided. It is sensible to split such plans into rectangles, with a crumple zone (or construction joint) in-between.

- The structural framing system chosen should invariably be of the “strong column and weak beam” type

- ***All buildings should invariably be designed to prevent collapse and loss of life under the most severe earthquake it is likely to be subject to within its design life.*** All structural components should be designed with adequate ductility, which would allow large plastic deformations to develop, without significant loss of strength or structural integrity.

The lessons from this experience and loss of life must be an eye-opener for all building professionals. Sound engineering principles should never be compromised and there is no room for complacency, when it comes to safety.

14.0 SUMMARY

In recent years, case studies have come to be recognised as a source of understanding our present state of technology and its limitations. Much improvement of our design concepts has been possible from a study of failures; these provide an invaluable source of information about design limitations. Design is a process of the anticipation of failure, and as such the more knowledgeable the designer is about failures, the more reliable his designs will be.

This chapter provides examples of failures due to design error, construction error and communication gap among the team members having different responsibilities.

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