Confidential Reporting Of Structural Failures And Lessons Learnt

NEWSLETTER

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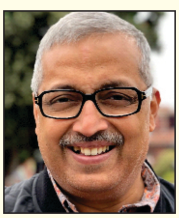
FROM THE DESK OF THE PRESIDENT

I am happy to note that the editorial board of CROSFALL is coming out with the sixth edition. The objective of this unique newsletter is to educate the readers about structural failures or near-miss without revealing the identity of the person or the project. It has been well received by the readers and helped them to be cautious in their respective projects.

Every edition of CROSFALL goes through a rigorous review process. Editorial board members & domain experts are doing fantastic work in evaluating, editing & reviewing the reports. The current issue contains reports which raise serious concerns on various aspects such as failure/distress of post-tensioned I girder during prestress, failure of steel bridge during cantilever launching and deterioration of RC stadium during construction.

It is noticed that gradually people are coming forward to send the reports. I urge civil & structural engineers to send reports freely without any fear or hesitation. Reports may be for any type of structural failure or structures with visible gross structural deficiencies and substantial risk of failure. Do send your feedback & suggestions.

— Prof. R. Pradeep Kumar



MESSAGE FROM CHIEF EDITOR

This is the 6th issue of the digital newsletter CROSFALL (4th in the current year). In this newsletter, there are three failure reports presented, which includes reports from building failure, report from a bridge failures during erection and launching and a report of a precast girder failure caused due to poor design & detailing. Each of these reports are stripped of identifying marks. Expert opinion is added in the reports to demonstrate how failures could be avoided and how safety could be improved.

CROSFALL has the unique distinction of being the only newsletter in India that collects and disseminates lessons from structural failures, design or construction errors, near misses, mishaps and related safety issues. CROSFALL is a powerful newsletter for structural engineering fraternity. It helps to mitigate risk of structural failures and thus safeguard the public and themselves.

I appeal to each and every reader of this newsletter to come forward and report about their experience on near misses or failures. Sharing knowledge among the structural engineers is one of the ways to educate our fraternity and to prevent failures and collapses. It gives an opportunity to structural engineers to learn from others mistakes.

Happy Reading!

— Alok Bhowmick

Contents

REPORT No. CF-19

Failure/Distress Case Study of Post Tensioned I Girder during Prestress and Lifting operation at Construction Stage

The reporters in this case are structural engineers, reporting cases of failure/distress in Precast post tensioned I-Girder during construction. 2 case studies are presented in this report

1. Introduction

Precast Post-Tensioned I Girder is a very commonly used superstructure system which is widely used in Highway Major/Minor Bridges and viaducts. It is a simple type of structure where the Girder is cast in the yard and Pre-stress operation is carried out, when concrete attains the specified strength at the time of stressing based on Grade of concrete after 5 to 7 days of casting.

This report presents case studies of unsatisfactory performance of Post tensioned 'I' girder (span > 30m) during prestressing operation and failure during lifting operations.

2. Case Studies

Two case studies have been presented to illustrate unsatisfactory performance and failure of Precast Prestress I-Girders. Generally, lateral bending is quite common in the prestress girders during prestressing, due to unsymmetrical tendon layout and prestressing force. To avoid such issue, layout of prestressing tendon has been kept symmetrical for both these case studies.

2.1. Case Study - 1

The outer girders have been proposed as unsymmetrical w.r.t c/l of web (as shown in Fig 1, 2 & Fig 3 below) for functional/aesthetic reasons. The length of girder is about 34.8m to be placed for 35m overall span of the superstructure. The prestressing layout is proposed symmetrical to vertical axis of the girder and plan deflection of girder has been checked in model during design, which is nominal. (Refer Fig 4).

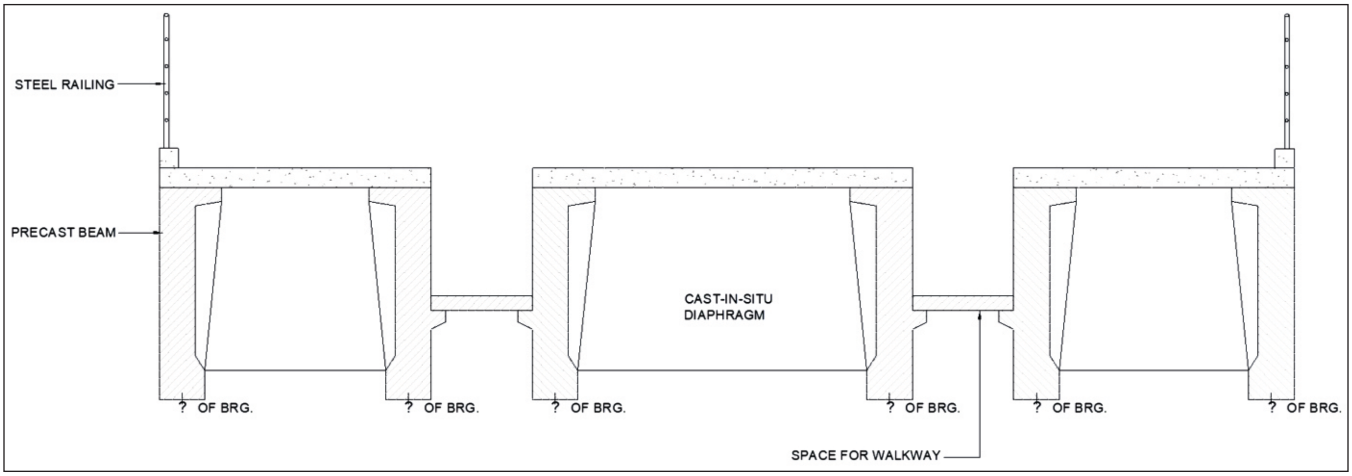


Fig 1: Cross Section of Superstructure

After prestressing of outer girder, lateral deflection was observed as 27mm, which, although higher than calculated deflection but not alarming. During lifting of girder from casting yard, there was sudden increase of lateral deflection (up to 300mm) with rotation about the axis of the girder (Photo 2). The construction team had kept the girder at original support without shifting due to excessive rotation. After sometimes, girder had been lifted again for shifting, girder rotated about 70 degrees to its axis and suddenly broke @3L/8 location (Photo 1). Fortunately, no casualty happened during this operation.

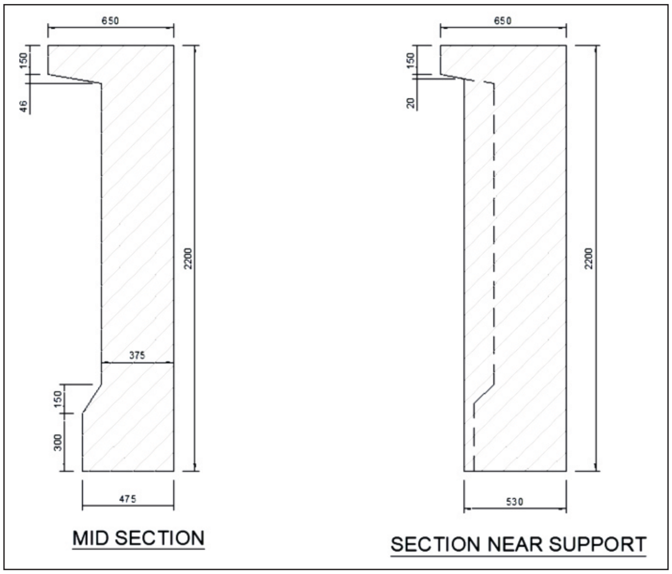


Fig 2: Cross Section of Girder

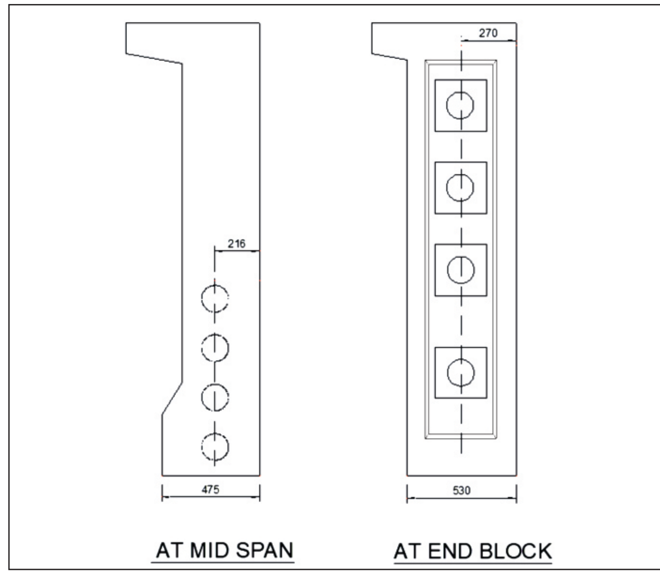


Fig 3: Section Showing Tendon Locations

The probable reason of the failure as envisaged, is listed below:

2.1.1 Construction Defects :-

a) Geometrical imperfection during placing of Tendon at site: Tendon placed eccentric w.r.t position shown in approved drawings (in plan).

b) Excessive horizontal buckling about Minor axis: Girder buckled much more than anticipated during design due to slender behavior.

c) Improper lifting arrangement: Centre of gravity of lifting hook didn't coincide with centre of gravity of self-weight of girder.

2.1.2 Design Deficiency :-

a) Unsymmetrical girder section - Led to buckling in minor axis.

b) Buckling Check during erection of girders, was not considered by the designer during detail design.

c) Slenderness check of I Girder might not have been carried out properly, refer calculation below.

As per clause 11.4.2 of IRC 112 (Slenderness limits of Beams), it is stated that "To ensure lateral stability, a simply supported or continuous beam should be so proportionated that the clear distance between lateral restraints does not exceed 60b or $250b^2/h$, whichever is the lesser, where 'h' is the effective depth to tension reinforcement; and 'b' is the breadth of the compression face of the beam midway between restraints."

In this case, h = 1890mm; b = 425mm (considering bottom flange since compression flange during construction will be bottom flange)

60b = 60*425 = 25200mm; 250b^2/h = 250*425^2/1890 =23893mm.

Lesser is 23893mm <34800mm.

Designer might have considered this check for top flange considering top flange as compression during service stage.

Photo 1: Girder Break at 3L/8 Location during Lifting

Photo 2: Plan Bending after shifting of girder from Casting Yard

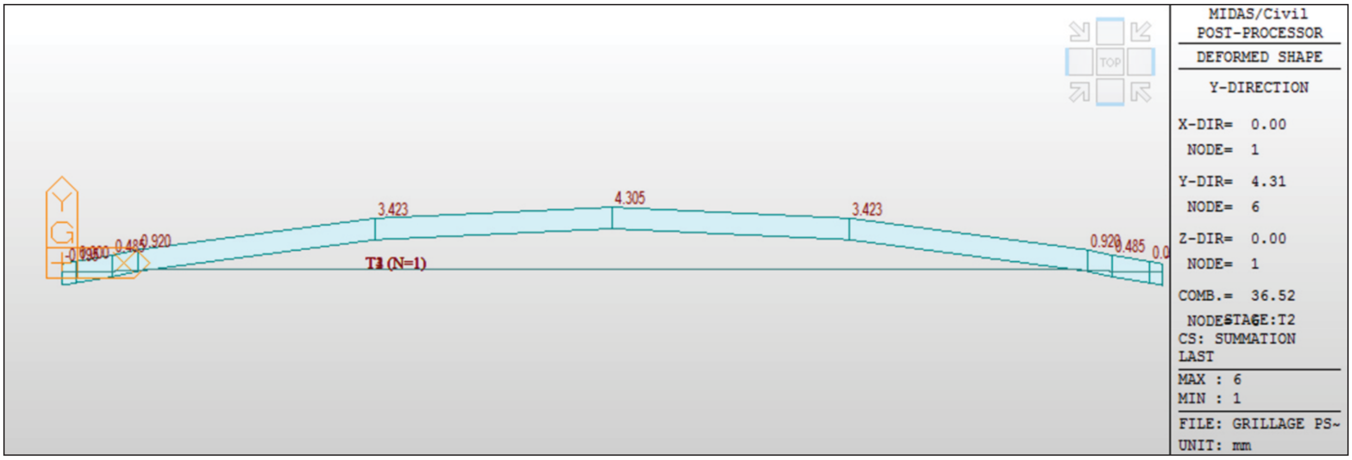


Fig. 4: Lateral Deflection of Girder as per Midas Analysis (only 4 to 5mm) lateral Bending

In the revised design, girder sections had been modified with symmetrical girder & increased bottom flange width to solve the issues in all future girders.

2.2. *Case Study - 2*

The second case study is related to excessive plan bending of precast girders after prestressing operation at casting yard. The length of girders was about 34.5m to form 35m superstructure span. The cross section of the superstructure and girder including tendon locations, has been shown in sketch (Fig 5 to 7) below.

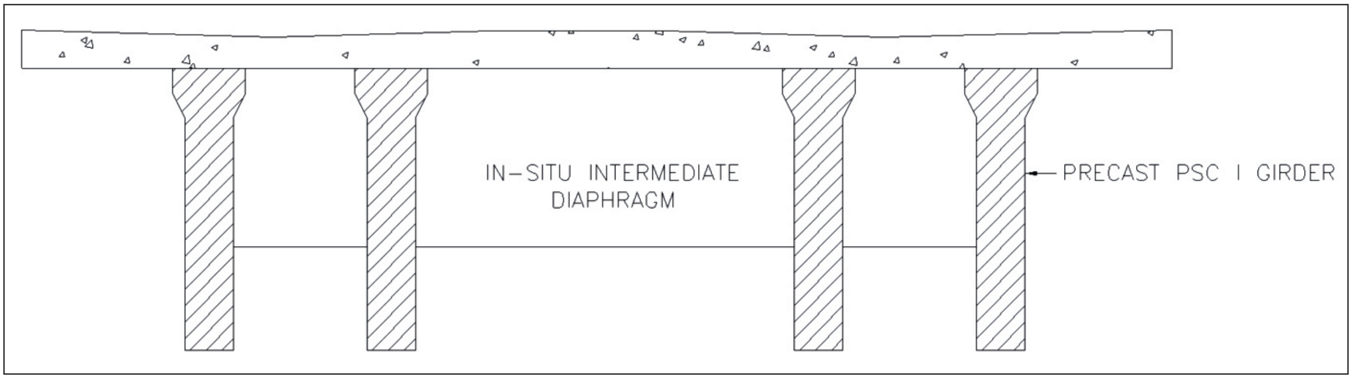


Fig 5: Cross Section of Superstructure

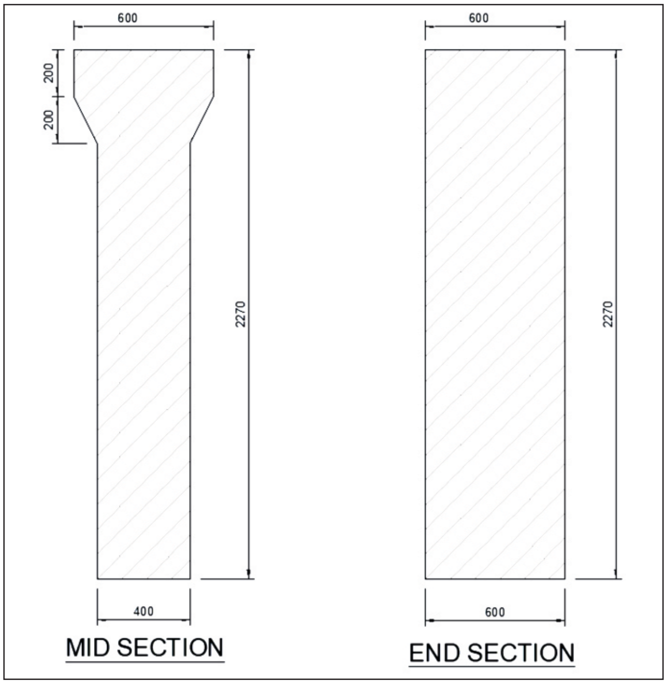


Fig 6: Cross Section of Girder

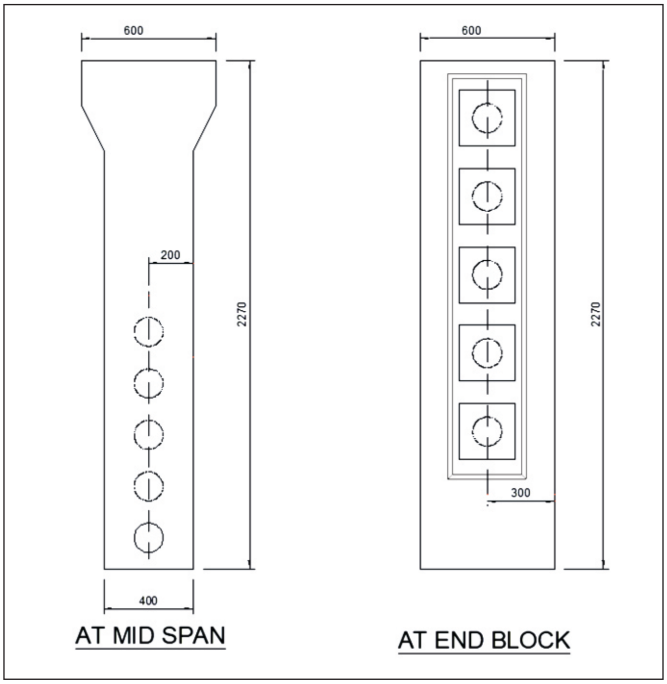


Fig 7: Section Showing Tendon Locations

The prestressing operations had been proposed in two stages in casting yard with symmetrical cable profile and forces. In this case, all the four girders (of the span) had excessive lateral deflection after two stages prestressing and lifting from casting yard to stacking yard. The value of lateral deflection was quite high varying from 100mm to 160mm.

2.2.1 The possible reason of the lateral deflection may be as below:

a) Slender beam: Slenderness check might have not been carried out for top flange as explained in Case-A.

b) Geometrical imperfection during placing of Tendon at site: Tendon placed eccentric w.r.t. position (in plan) shown in approved drawings.

Design had been reviewed with as-built lateral deflection (considering eccentricities of cable due to imperfection) and it had been observed that there will be permanent tension at one corner of the bottom flange of the girder during service stage. A proposal had been submitted with external prestressing through diaphragm to control the tension in bottom flange. But the proposal had not been executed due to client's reservation. The precast PSC girders have been replaced with new I-girders with modified cross section.

3. Lessons Learnt

Unsymmetrical section of Post tensioned 'I' Girder generally should be avoided. The design check of such type of structure requires rigorous 3-D finite element analysis through software only to assess the true behavior of structure. Special attention shall be taken for dimensioning of precast I-girder to avoid buckling/lateral deflection during construction.

To avoid such type of incident in future the following remedial measures may be adopted:

a) Slenderness and lateral stability check must be performed for both symmetrical and unsymmetrical types of Post tensioned 'I' Girder. The dimension of bottom flange to be increased to avoid a slender beam.

b) As per code, there are limitations of compression flange of 'I' beam. Most of the designers perform the check for the top flange of girder considering compression flange. However, post tensioned I-beams is very critical in construction stage with compression at bottom flange. Hence, minimum dimension of bottom flange width shall be maintained to avoid a slender beam section. This may be clearly mentioned in the code to avoid ambiguity among designers.

c) The moment of inertia of Post tensioned I Girder about minor axis is very low, therefore lifting check and lifting location to be defined carefully so that during erection no lateral tilt happen.

d) Special care to be taken for lifting of unsymmetrical beam with special arrangement after design check.

e) A lifting methodology to be prepared by contractor in consultation with designer to avoid any miscommunication between designer and site engineer.

f) At site cable profiling is to be done carefully to avoid any lateral eccentricity during pre-stressing operation. Designer shall perform design checks with maximum allowable tolerance for cable laying.

g) Lateral deflection during and after prestressing operation to be monitored. The lateral deflection before and after prestressing shall be tabulated, the same shall be brought to designer for acceptance.

Comments of Expert Panel

The failure reported in this case is not uncommon and has occurred in several projects in the past. Such failures can be avoided by adequately dimensioning the top and bottom flanges to avoid such failures.

The importance of checking for lateral instability of symmetrical and unsymmetrical, Precast Prestressed I-Girders cannot be overemphasized. Out of plane buckling about the minor axis must be guarded against during both Prestressing and lifting operations. In addition, the importance of correct cross section dimensioning to avoid lateral buckling is also important. The designer must also check that the Cable profiling at site, prior to concreting, is within tolerance limits to avoid any tendency for lateral buckling.

Post tensioned I-Beams can develop compression in bottom flange during construction therefore designer must take this into account and increase the dimension of bottom flange suitably.

The reported failure in this case gives an impression that there was no serious peer review/proof checking by experienced and seasoned bridge engineers. Unfortunately adequate provisions are lacking in IRC/IRS codes to prevent such practices. In order to avoid such failures in future, IRC / IRS Code should be amended.

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REPORT No. CF-20

Failure of Steel Bridge with Open Web Girder (OWG) during cantilever launching

This report is regarding the failure of a steel through type open web girder (OWG) bridge during the cantilever launching. The failure happened when the cantilever span was being launched by the process of cantilever method. The whole span was teared from the anchor span during the launching process and as a result these were casualties.

1. Salient details of the bridge and the launching scheme

The bridge comprised of multiple spans of varying span length and supported on piers of varying height. The overall span configuration is 2x33.4m steel composite girder + 1x106.0m OWG + 1x 49.74m OWG + 1x106.0m OWG + 1x49.74m OWG.

The bridge was designed for cantilever launching method of construction for the OWGs. The cantilever launching was started from abutment A1 side. One span of 103.5m was assembled partly on the embankment and partly on the A1-P1 and P1-P2 span. The span between P2 and P3 was supposed to be cantilevered from the anchor span of 103.5m span, erected beyond P2 towards A1. Fig. 1 shows the elevation of the bridge and Fig. 2 shows position of cantilever span during launching.

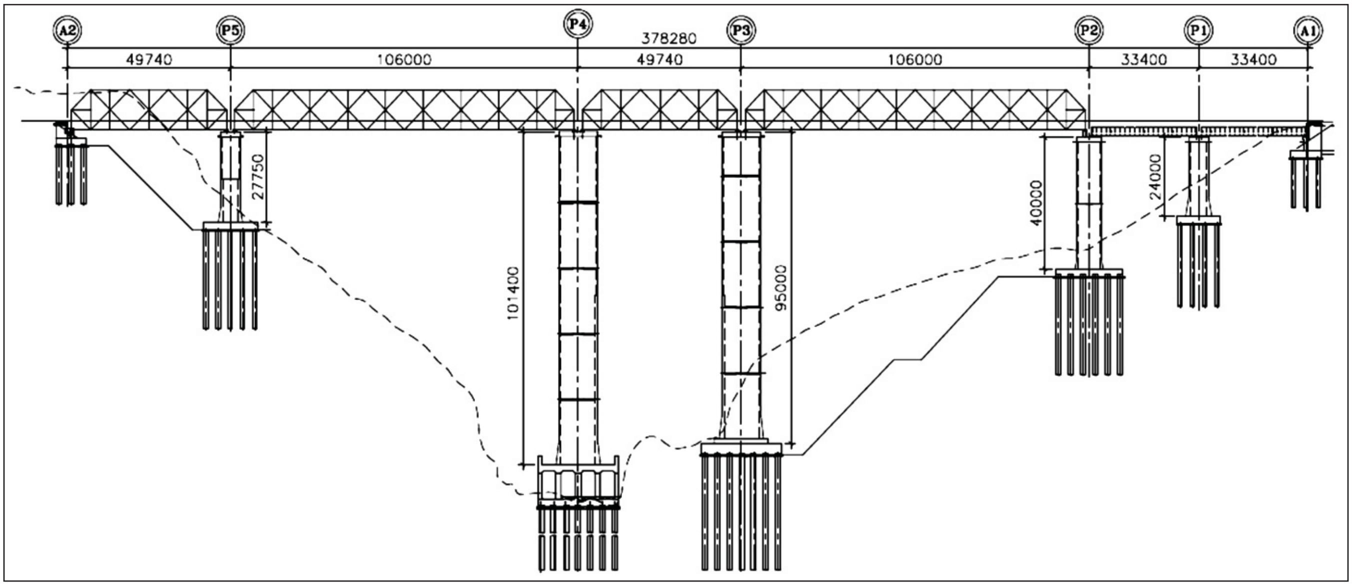

Fig. 1 : Elevation of the bridge

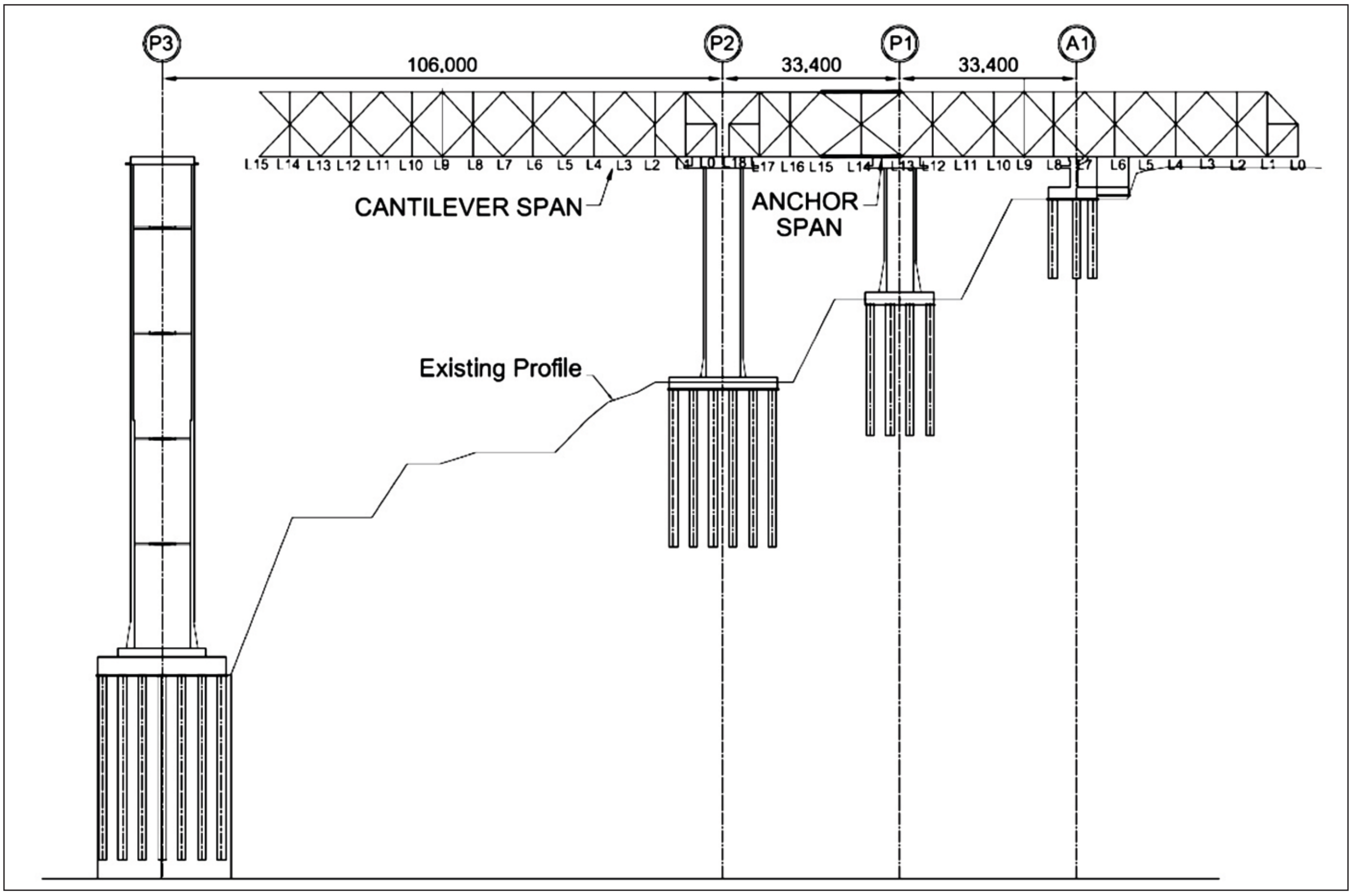

Fig. 2 : Position of cantilever span during launching

2. Failure during launching

The anchor span of 103.5m was originally designed for cantilever launching with respect to another 103.5m span. There were total 18 numbers of panels (L0-L1 to L17-L18) of 5.75m length each in each span. However, during the erection of the anchor span, the contractor modified the anchor span connection detail in order to keep one node point of the truss on pier P1 which was not any essential criteria. In the process the two panels (L13-L14 and L14-L15) of the OWG had length of 7.7m. Fig. 3 shows the original span configuration and Fig. 4 shows the revised span configuration of the anchor span fabricated during cantilever launching.

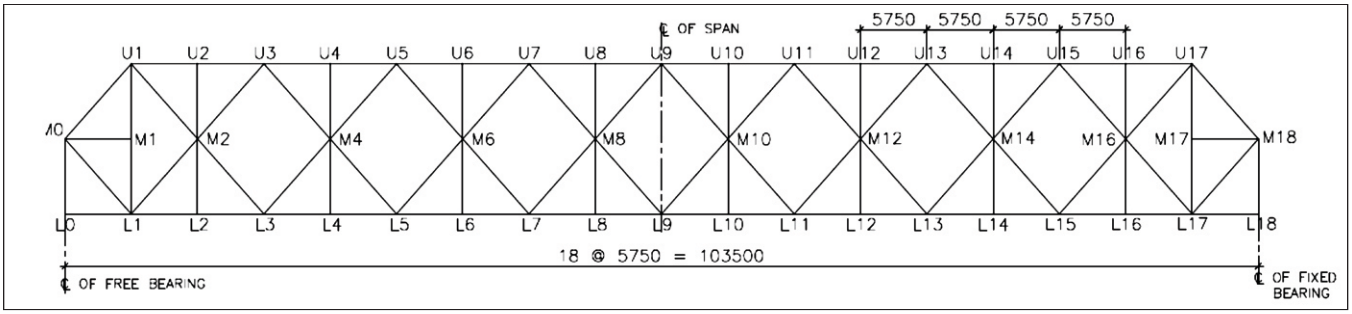

Fig. 3 : Original span configuration

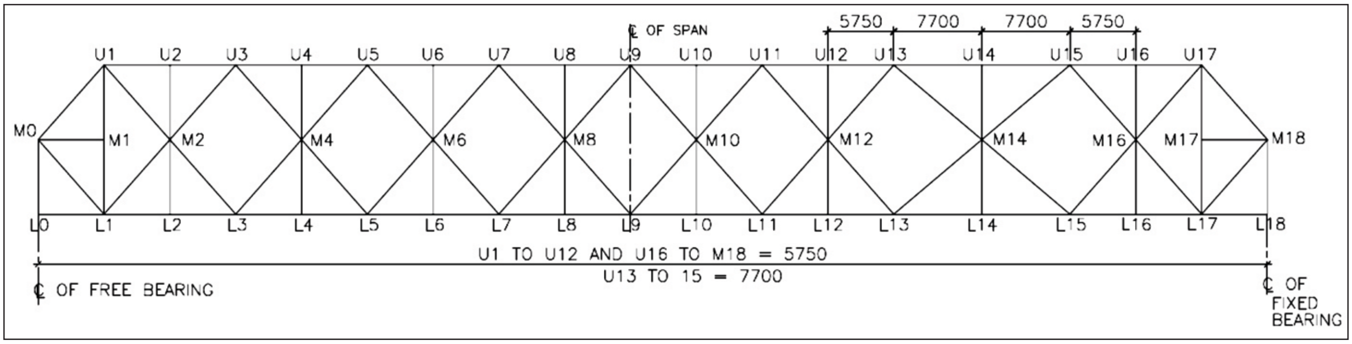

Fig. 4 : Revised span configuration of the anchor span fabricated during cantilever launching

These new panels were of same section which was originally designed. The failure occurred when cantilever launching was progressed between P2 and P3 span up to the node point L15. There was a sudden snap occurred at U14 joint and as a result the entire cantilever span along with part of the anchor span between node L14 and L18 was separated from the anchor span and fell on the hill slope (Refer Fig. 5).

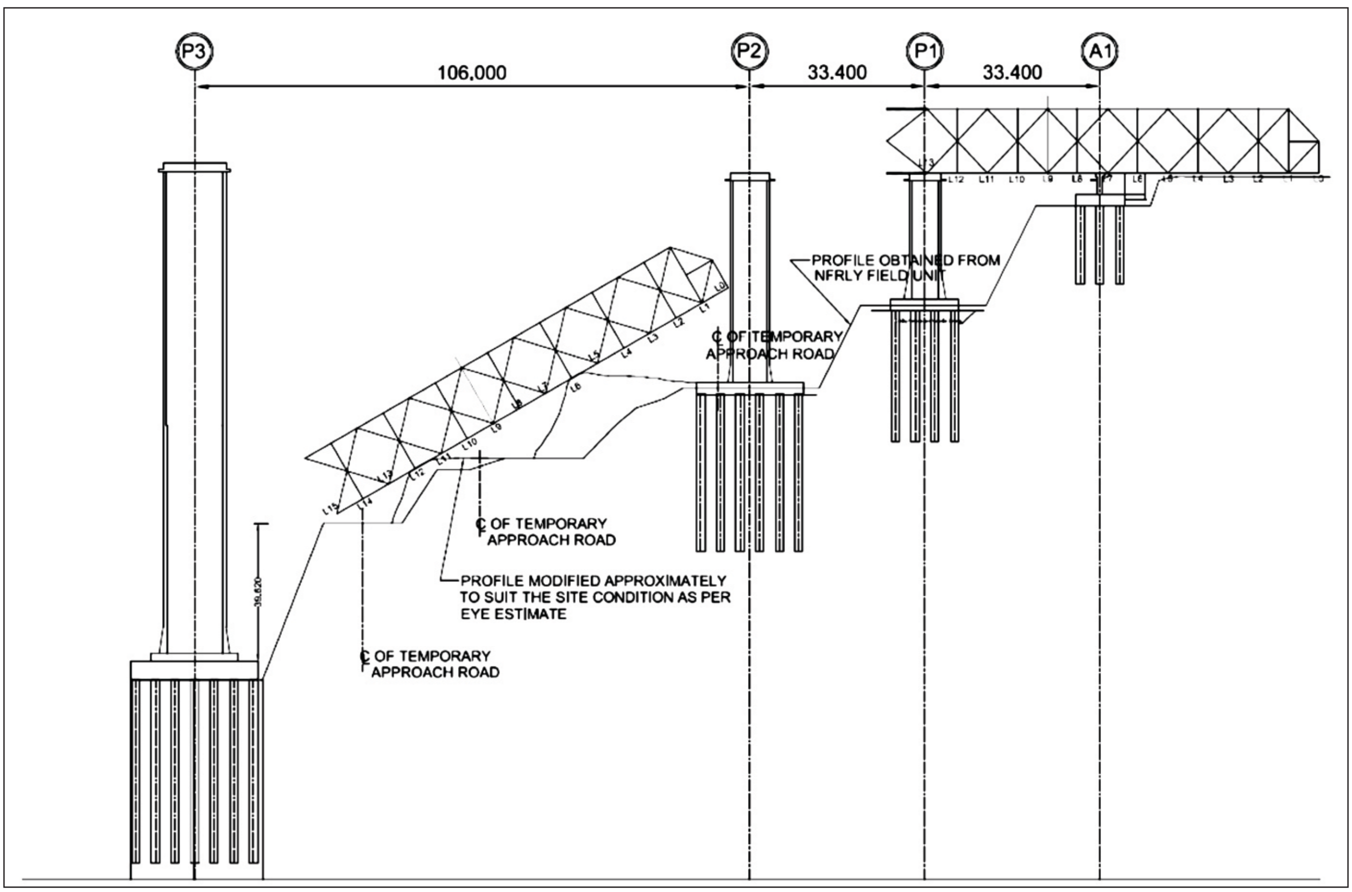

Fig. 5 : Fallen cantilever span

Fig. 6 shows the failure point in truss. It is observed that the panel U13-U14 and U14-U15 provided are of 7.7m length each and they are not connected by a full strength splice. The splice provided at the joint is found to be adequate to cater for the connection forces of vertical members but it was found grossly inadequate to transfer the tension force from U13-U14 to U14-U15 (Refer Fig. 4).

Fig. 6 : The snapped anchor span – the snapping joint at U14 shows no full-strength splice

3. Possible Cause of failure

Fig. 7 shows the splice details of the U13, U14 joints for the originally designed span. Under the revised condition, the splice at U14 should have been similar to the splice at U13, which is a full-strength splice capable of transferring tension force from U12-U13 to U13-U14. However, the failed splice is kept similar to that of splice at U12 joint where the panel U11-U12 and U12-U13 are single member of 5.75x2 = 11.5m length and the corresponding splice is only to transfer load from the vertical member to the joint. Had there been a single member between U13-U14 and U14-U15 of total length of 7.7x 2 = 15.4m, the requirement of full-strength splice to transfer the tension force from one panel to the other could have been avoided. However, due to non-availability of the plate of 15.4m length this was not done and at the same time the full strength splice was also missing which caused the failure

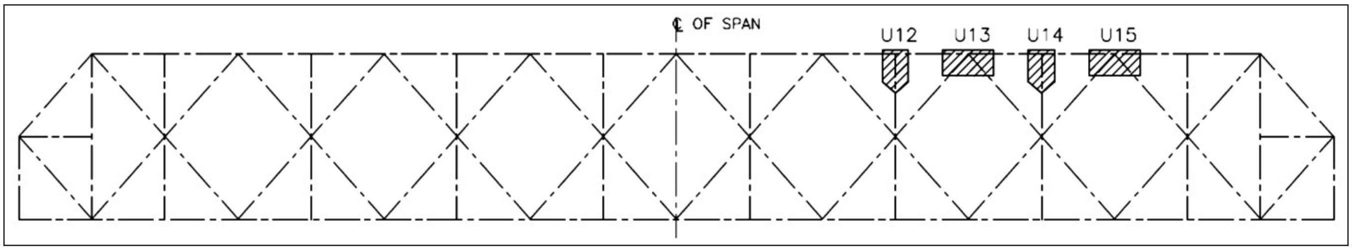

Fig. 7 : Splice at U13 and U14 joint in the original span designed for cantilever launching

4. Lessons learnt

The following lessons are learn from the above failure:

a) 'Construction Stage' design is equally important (if not more), as composed to 'Service Stage' design

b) No design change shall be permitted during construction, without bringing if to the notice of design Engineer. The design change, if needed, should be cross checked thoroughly and if necessary independent design checking should be done

c) Detailing is extremely important particularly for a steel structure and this should be checked by an experienced engineer

d) Knowledge of availability of material is important. In the present case, the designer may have thought that the member may be fabricated over a length of 15.4m by using plate of the similar length which does not lead to the requirement of a full strength splice to transfer direct tension. In practice, due to non-availability of the 15.4m length of plates, separate members of 7.7m length have been fabricated which required full strength splice to transfer direct tension from one member to the other, which was not done.

e) Fabrication drawing prepared based on design drawing should also be checked thoroughly by experienced detailer.

5. Comments of Expert Panel

In this reported case of failure, certain design changes were made by the contractor without referring it to the designer and without checking the splice design. It is clear that the Contractor did not have the competence necessary to design and detail the changes.

Steel structures are very sensitive to detailing and erection apart from design and many failures have occurred when these aspects are neglected and not supervised properly. Once the GFC drawing is issued to the Contractor for execution, Client should ensure that no design changes are permitted in the drawings and in execution without the involvement of original designer.

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REPORT No. CF-21

Deterioration of Reinforced Concrete Stadium Building during Construction

1. General

This report pertains to the design deficiency & structural deterioration leading to an imminent failure of a stadium building located in the northern region of India. Fortunately, prompt intervention by the sports authorities averted a potentially catastrophic incident.

The failure of structures during construction is a significant concern in the construction industry. Such incidents can result in loss of life, property damage, and can be economically costly. The structural collapse/failure may occur due to the following reasons as shown in Fig. 1.

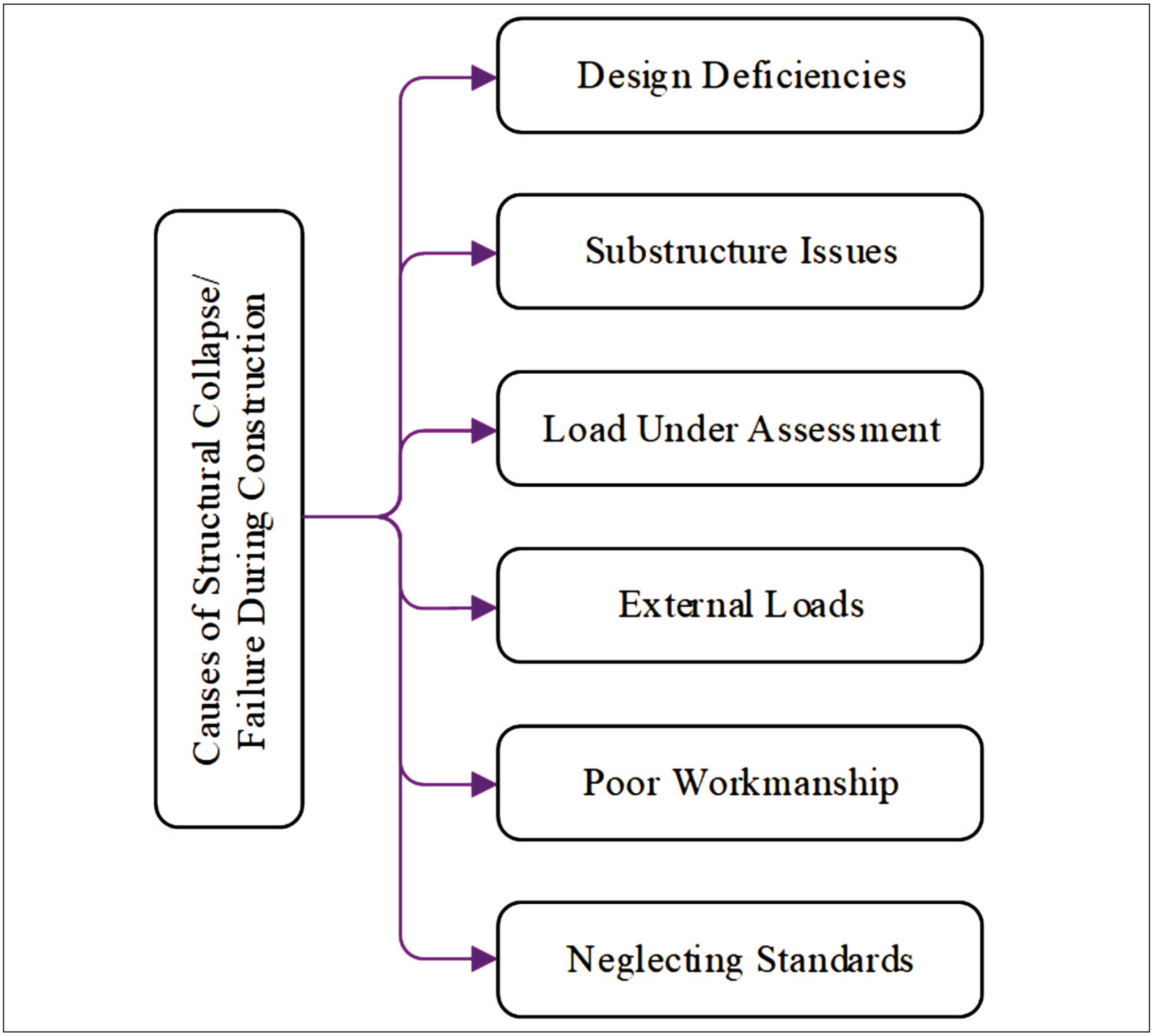

Fig. 1: Failure of structures during construction

2. Details of Stadium Building

The sports stadium building is a RCC frame structure. So far the RCC work has been done up to the pavilion floor level. The construction work of frame was executed during the year 2010-11 and 2011-12. The finishing work of this building is yet to be carried out as the structure is partially constructed. The structure at present is still lying as such and is being subjected to weathering over the years.

The structure is a RCC frame structure, comprising of beams, columns and slabs. It consists of four levels as seen from Fig. 2: (a) Plinth level (b) First floor level (c) Mezzanine level (d) Pavilion level. The roof is at a level of 43'' 3' from the Ground level. It lies in the Zone IV according to the seismic hazard map of India given in IS: 1893-2016 and hence the location is prone to severe seismological activities. In addition, as the stadium is an open structure, the wind load is an important load to be accounted for in the structural analysis.

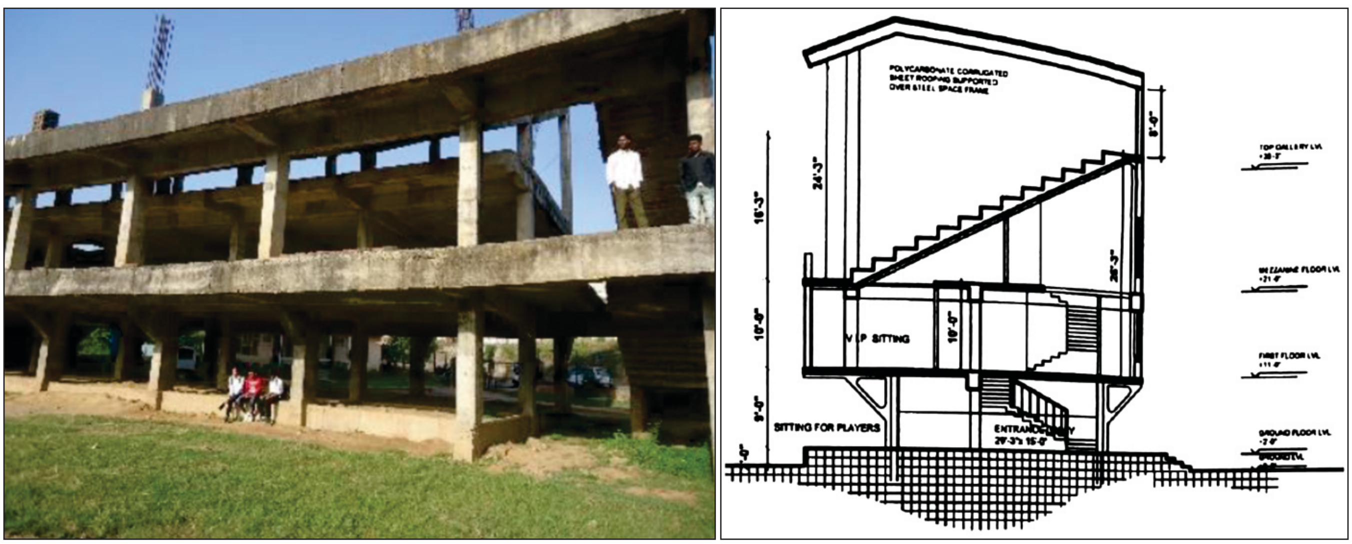

Fig. 2: Different levels of the Stadium

The signs of structural deterioration in the building are elaborated as listed below, and can be visualized from Fig. 3.

2.1. *Plinth Beam:*

- Plinth beam not properly cast.

2.2. *Cosmetic Finishing and Repairs:*

- Signs of cosmetic finishing and repairs over possible underlying cracks in concrete.
- Poor finishing surface of concrete at many places.
- Signs of using kuccha (unrefined) shuttering in some areas.

2.3. *Reinforcement Issues:*

- Casting work carried out without proper placement of chairs for reinforcement which resulted in improper concrete cover, exposing reinforcement to the atmosphere and is causing corrosion.
- Bond between concrete and steel bars compromised.

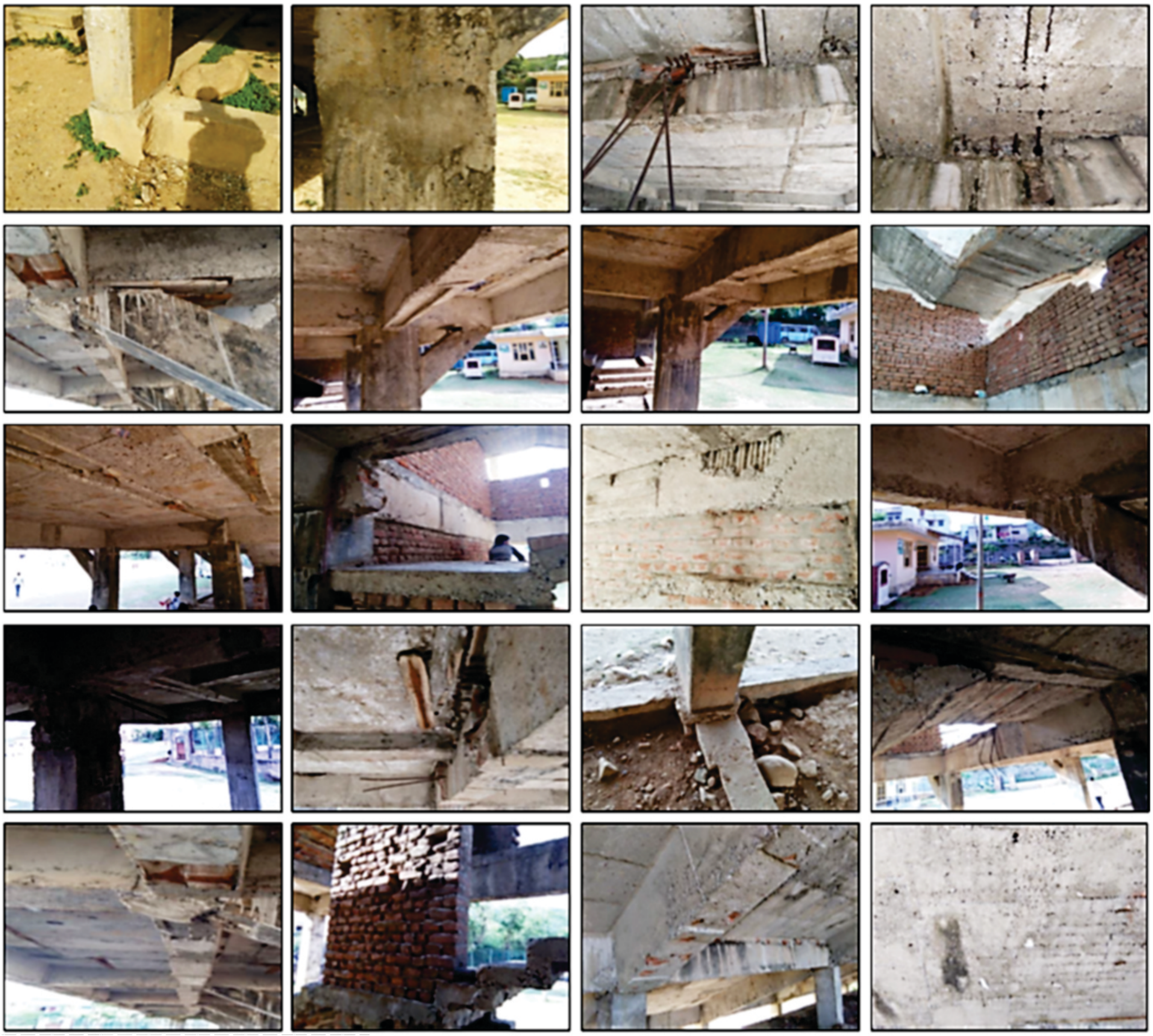

Fig. 3: Signs of deterioration

- Irregular shape and size of reinforced concrete beams observed.
- Exposure of reinforcement bars in beams, leading to corrosion.
- Plinth beam improperly casted with non-uniform cross-section.

2.4. *Inclined Beams and Staircases:*

- Inadequate casting of inclined beams.
- Cosmetic repairs and filling done afterwards.
- Improper support from inclined beams to the structure above, causing eccentricity of loads.

- Required beams for structural reasons missing.
- Beam near stair-case landing removed/dismantled, posing potential structural implications in future.
- Lack of end-portion in inclined beams, affecting their support function.

2.5. *Alignment and Structural Deficiencies:*

- Beam alignment issues.
- Different phases of concreting works in columns not carefully carried out.
- Structural cracks observed in reinforced concrete beams.
- Concrete issues, including honeycombing, exposed reinforcement, and poor workmanship.
- Substandard masonry work with excessive mortar thickness and English bond not maintained.
- Irregularity in reinforced concrete beam depth.
- Sunken floor corners not properly aligned.

2.6. *Foundation and Structural Integrity:*

- Foundation problems with irregular geometry.
- Missing joist/beam impacting landing placement.
- Cosmetic repair patches on concrete.
- Embedded external materials in concrete affecting cross-section.
- Non-uniform cross-section of cast in-situ beams.
- Structural cracks in columns causing reinforcement to be exposed.

2.7. *Roof and Roof Slab:*

- Sagging in the roof slab.
- Uneven concrete surface finish.
- Thickness of suspended floor slab is uneven.
- Structural detailing deficiencies in columns.
- Hairline cracks in reinforced concrete beams at mezzanine floor.

2.8. *Electric Conduits and Pipes:*

- Electric conduits and pipes exposed, causing material deterioration.

3. Results of NDT and Structural Analysis

3.1. NDT

Based on the Rebound Hammer tests, the estimated compressive strength of concrete in columns varied from 17 to 28 N/mm2 and in beams it varied from 24 to 28 N/mm2. However, it is observed that the estimated compressive strength of the concrete used in plinth beams is in the range of 23 to 29 N/mm2 on the Ground Floor. The estimated compressive strength of the concrete in footing is around 27 N/mm2. On the first floor, the estimated compressive strength of concrete in columns varied from 22 to 29 N/mm2 and in beams it varied from 25 to 32 N/mm2. The estimated compressive strength of the concrete in roof slabs varied from 24 to 30 N/mm2. On the second floor, the estimated compressive strength of concrete in columns varied from 23 to 30 N/mm2 and in beams it varied from 22 to 31 N/mm2. The estimated compressive strength of the concrete in roof slab varied from 32 to 35 N/mm2.

The quality of concrete was Medium to Doubtful in most of the columns and beams based on UPV tests, however, only few beams were showing the quality 'Good' on the First Floor. The probability of corrosion in all the RC components was less than 50% to 90% based on the results of corrosion analyser.

3.2. Chemical Testing

The pH values of concrete in structural elements were within the safe limit. The amount of total chlorides in the concrete samples being in the range of 0.011%-0.014% i.e. was less than the prescribed tolerance limit of IS: 456-2000. The Sulphate content in the sample was in between 2.2% - 6.3% which is not within the specified limits as per IS: 456-2000 i.e. < 4.1%. The increase in presence of sulphate in concrete will cause expensive crackly of the concrete matrix and will further attracts the probability of corrosion in the embedded reinforcement in the concrete.

3.3. Structural Analysis

Structural analysis and design was also checked for different load combinations as per current practices of design code (IS: 1893-2016). The results showed that the design of the structure was insufficient. Surprisingly, various structural members of the structure were not designed properly. Some of the structural members needed to be redesigned, as the sections provided were lesser than required. In several members, reinforcement provided was not sufficient to fulfil the design needs. It was concluded that the high ambitions and requirements of the architect designer was not carefully incorporated by the structural designer.

It was therefore inferred that there was a need for redesigning of the entire structure as per design guidelines in the Indian Standard codes. In addition to these, the structural dynamic analysis of the sports stadium building structure under crowd loading must have been performed, in order to ensure that resonance does not occur when the stadium is loaded to its full capacity.

The structural design, workmanship and quality of the construction work of stadium building was not observed up to the mark. Further the structural auditor advised that any further over loading, construction activity or movement on the existing structure is not advisable. The risk might be of very high magnitude in the event of any earthquake. It was suggested to redesign the structure strictly as per the latest codal provisions and recommendations. The consideration of dynamic analysis for resonance is also mandatory considering the importance of the building. The plan of ground first and second floor plan is shown in Fig. 4.

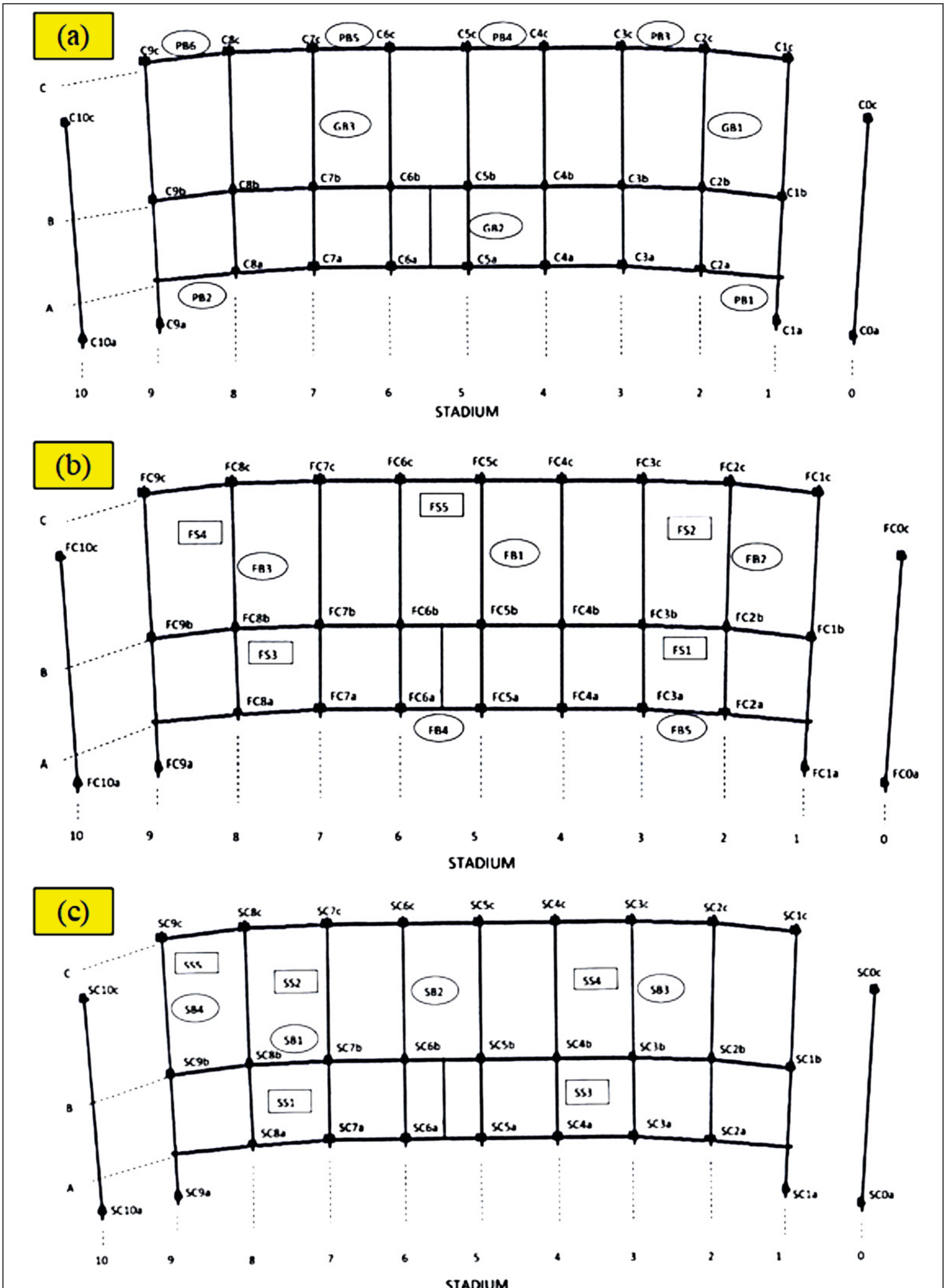

Fig. 4: Plan of (a) Ground, (b) First and (c) second floor plan

4. Lessons Learnt from this failure case

(i) It is the negligence of the structural designer as well the engineer responsible for vetting the drawings, which led to the structural failure of the building structure.

(ii) The structural designer should have revised the typical architectural requirements in consultation with the architect considering his limitations in designing particularly the dynamic analysis.

(iii) The structural designer must design any kind of structure irrespective of its importance as per the latest available codes of practices especially the structural ductility detailing in a high seismic zone.

(iv) The site engineers and concerned government officials should frequently visit the site during construction and ensure the quality of the construction materials and importantly the proper workmanship.

5. Rectifying the issues regarding the stadium building

(i) Immediate Structural Assessment: Conduct a thorough structural assessment to identify critical issues and prioritize areas requiring immediate attention.

(ii) Redesign and Retrofitting: Redesign and retrofit the stadium's structural elements to meet the current design code standards, especially for seismic and wind loads. Ensure proper reinforcement and section sizes.

(iii) Quality Control: Implement strict quality control measures during construction, including proper placement of chairs for reinforcement, formwork, and concrete quality checks.

(iv) Dynamic Analysis: Perform dynamic analysis under crowd loading to prevent resonance and ensure the structural integrity of the stadium during events.

(v) Corrosion Prevention: Implement measures to mitigate corrosion risks, such as addressing the elevated sulphate content and ensuring proper concrete cover for reinforcement.

(vi) Regular Maintenance: Establish a regular maintenance program to address weathering and wear and tear over time, ensuring the long-term structural integrity of the stadium.

(vii) Training and Education: Train construction personnel and engineers regarding the importance of adhering to design standards and quality workmanship.

(viii) Compliance with Codes: Ensure strict compliance with the latest Indian Standard codes for structural design and construction.

6. Comments of Expert Panel

The case discussed in this report is not strictly a non-repairable failure which leads to demolition/collapse, but discusses the advancing deterioration and structural deficiencies in a structure which is incomplete and yet to be put to its intended use.

Whilst being generic in its various observations and corrective measures suggested, the report also suggests various measures to be taken a new to repair and retrofit the stadium structure and its Grand stand due to insufficiency in designs as well as non-addressing of Crowd loading and pedestrian induced vibration as part of the design requirement. The structure is to be retrofitted to the latest requirements of the various Indian Building and seismic codes.

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About the CROSFALL Newsletter

CROSFALL is a newsletter created by Indian Association of Structural Engineers (IAStructE). Its purpose is to share lessons learnt from structural failures, near-misses and safety concerns. CROSFALL is greatly encouraged and inspired by CROSS (Confidential Reporting on Structural Safety), UK, which is a collaborative effort of three institutions (IStructE, ICE and HSF). There is however no connection between CROSFALL-IAStructE and CROSS-UK.

CROSFALL has a confidential reporting system, which allow safety issues and failures to be reported by professionals, without exposing their identity. Any identifiable details, such as a project, product, individual or organisation, remain completely confidential to CROSFALL editorial team. Reporters' personal information will be collected to only verify the contents of the report, and to communicate with the reporter as and when necessary. The newsletter will report only failures and safety related issues with the objective to learn lessons from such failures and to help prevent future structural failures, by providing insight into root causes of such failures and spurring the development of safety improvement measures. CROSFALL team will depend on professionals to submit reports, whenever they can share their concerns about what they witness around or what they experience on any real-life projects. Anyone involved in the construction industry is welcome to submit a report. The more reports submitted, the better CROSFALL can identify and quantify safety issues across the industry. This will help the entire industry to learn lesson from CROSFALL publications

What can be reported?

- Structural failures,
- Poor Design and Detailing, Lack of Seismic Safety in planning
- Safety concerns about high risk erection schemes at Site
- Safety concerns on Temporary Works
- Near misses or observations relating to procedures followed at site, which may lead to failures or collapses.

To submit the report :

Visit : www.iastructe.co.in/crosfall.php

E-mail : crosfall.iastructe@gmail.com

Disclaimer :

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