

CASE STUDY: INVESTIGATING THE 2013 ASIACELL WAREHOUSE STEEL PORTAL FRAME FAILURE

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ABSTRACT

In a windy morning of March 17, 2013, a warehouse belonged to Asiacell Telecom collapsed during construction. The structure was a relatively large 108m by 48m double-span steel portal frame designed by an Iranian engineering firm and constructed by a local construction company. This catastrophic structure collapse that killed three workers and injured few, was a sudden failure that triggered an intense investigation to determine the cause to avoid the occurrence of the same mistake in the future.

This research paper is the product and outcome of the forensic structural analysis performed by the author for Asiacell that indicates the exact reasons behind the catastrophic failure. It provides the evidence to be a lesson-learned document for the future steel structure construction in Iraqi Kurdistan.

Keywords: Case Study, Steel Portal Frame, Structural Failure.

1. INTRODUCTION AND PROBLEM STATEMENT

This paper is a further investigation of the original analysis prepared to fulfill the request of Asiaccell, who owns the warehouse complex. The case was originally investigated and reported to Asiaccell in 2013 [1]. As such, an evaluation of the design and construction process started few days after the incident to determine the cause. The evaluation includes construction as well as the structural design. The structure (Figure 1) was located in Sulaimany in Kurdistan Region and was a part of a warehouse complex that consisted of a total of four portal frame structures besides few other buildings for the office. This frame, which was the largest one of the group, catastrophically collapsed during the construction process and triggered an investigation by Asiaccell to enhance the construction safety and insure that this kind of tragedies will not happen again. This paper depicts the structural analysis and design, and evaluates the construction management in order to highlight the cause of the catastrophic failure of the structure. As this type of failure is dangerous being a source of safety concerns and cause of losing money besides time consumption, Asiaccell demanded that work to be halted immediately and strong measures to be taken seriously through a thorough forensic analysis to prevent this type of failures in the future. The author of this paper was a part of the team of investigators hired by Asiaccell for this purpose and this paper is the result of the calculation and evaluation of the frame. As in any construction failure in the world, fingers were pointed to the design and construction process. As such, it was necessary to review the design documents first and determine whether there are any flaws in it that might caused the failure.

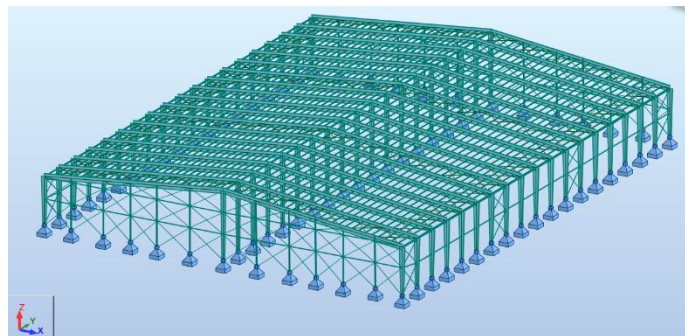


FIGURE 1. Asiaccell Warehouse Portal Frame Model

2. WHAT WENT WRONG?

The 108m x 84m steel portal frame collapsed during construction as a result of a dynamic lateral load caused by the collapse of the 50 ton rough terrain crane and hitting one of the rafters. The rafter that was connected to the outer columns and fixed on the concrete via four anchor rods, collapsed under the impact as one piece frame and hit the adjacent frame section and caused a catastrophic abrupt-action collapse of the whole structure as a domino effect (Figure 2).

The collapse was catastrophic as it killed three steel workers and injured few others. The crane failed under the strong wind while its telescopic boom was extended all the way. Despite that the load that the crane carried during the failure was well within its capacity, a question was raised as to why the construction was not halted during the high wind weather that was above 50 km per hour? Obviously, this was a flaw in the construction management of the project but then another question popped up: why the frame collapsed under the relatively-low impact load? This question is answered in the following paragraphs.

3. INVESTIGATION METHODOLOGY

The frame design was evaluated using Autodesk Robot Structural Analysis Professional 2013 software. The pitched roof steel portal frame was modeled to have the same member sections as the original design and was analyzed using the conventional LRFD method for steel structure. As the structure is a moment resisting and braced frame, special care has been taken to have members of tapered sections whereby the bigger sections are located at the location of bigger bending moment. Stiffeners have been provided to tackle torsion and lateral torsional buckling. The analysis is in accordance with ANSI/AISC 360-2013 [2].

The base plates were analyzed in accordance with AISC Steel Construction Manual, 9th Edition using Allowable Stress Design (ASD) [3] and following the procedures specified in the book Design of Welded Structures [4] and AISC Steel Design Guide 1 [5]. The anchor rods were analyzed in accordance with Appendix D of ACI 318-08 using LRFD Method [6].

The construction aspect was evaluated using a site visit investigation by checking the damages caused in the frame members, connections, and foundation. The observations were

photographed and illustrated in this paper. First, the design details are clarified and analyzed. Then, the construction process is investigated and the causes of the failure are concluded.

4. FRAME DESIGN DETAIL

The steel portal frame, which had a span of 84.0m and a length of 108m, was designed to have two spans and 18 bays (Figure 3). It was designed to be supported by side and intermediate columns on the direction of 84.0m span, which makes it to have two successive spans of 42.0m long (Figure 4). The frame possessed bays of 6.0m span on the traverse direction with column height of 10m on the sides and 15m in the middle. The structural steel was of Grade S275 according to Eurocode 3.



FIGURE 1. Collapsed Frame as Domino Action

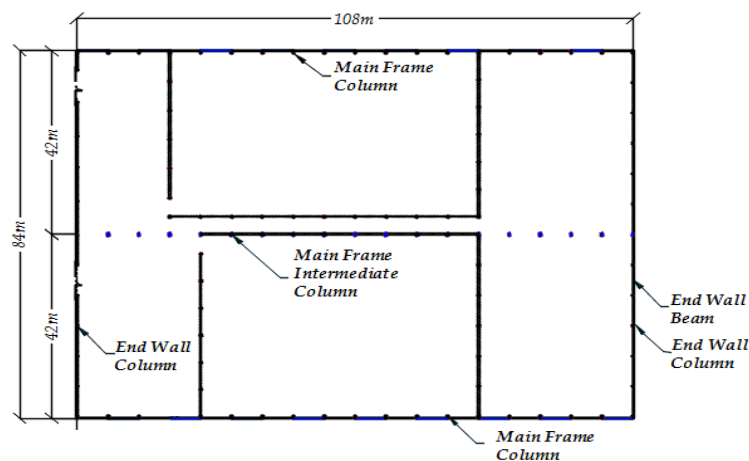


FIGURE 3. Asiatic Warehouse Frame Plan

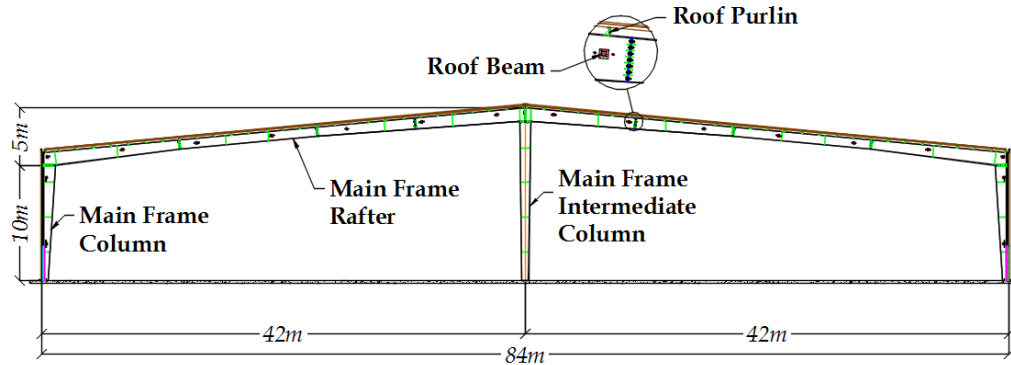


FIGURE 4. Typical Portal Frame Elevation

The frame consisted of the following main elements that were analyzed in detail:

1. Main Frame Columns: The main frame columns were designed to have tapered section on one side that were 600mm at the base and 1200mm at the top. The flanges had a width of 300mm and thickness of 16mm while the web had 10mm thickness.
2. Main Frame Rafters: The main frame rafter beams were designed to have four sections that were 1200mm at the column connections and at the pitch point and tapered for 12m to a constant section of 800mm. The flanges had a width of 300mm and thickness of 16mm while the web was 10mm thick.

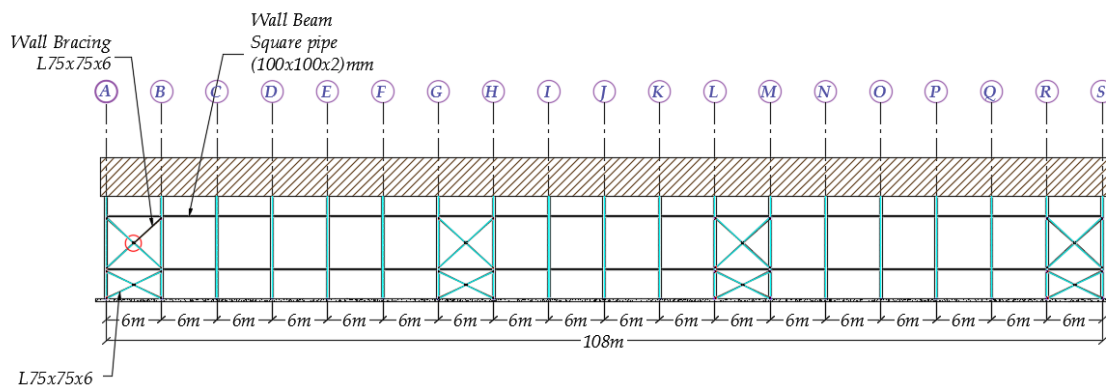


FIGURE 5. Portal Frame Side Wall Detail

3. Main Frame Intermediate Columns: The main frame intermediate columns were tapered sections on both sides that were 600mm at the base and 1200mm at the top and had the same plate thicknesses as the main frame columns.

4. End Wall Columns: The end wall columns were the standard sections of IPE 240.
5. End Wall Beams: The end wall beams were the standard sections of UAP 100.
6. Wall Bracings: The walls were braced in four bays with CAE 70x6 sections as shown in
7. Wall Beams: The wall beams were 100x100x2 box section beams.
8. Roof Beams: The main frames were tight together through 12 roof beams of steel tubes of 100x100x2 equally spaced.
9. Roof Purlins: The roof purlins were designed to be 175mm Z sections.
10. Wall and Roof Covers: The walls were designed to be covered with 50mm thick sandwich panels while the roofs to be covered with 80mm thick sandwich panels (Figure 6)
11. Roof Bracing: The roof was designed to have the bracing plan showing in Figure 7 with $\phi 16$ smooth steel bars.
12. Foundations: The columns were supported on isolated footings connected together with tie beams through the footing pedestals as shown in Figure 8.
13. Foundation Connections: The main columns were designed to be connected to the foundation through four $\phi 29$, 600mm long anchor rods that were embedded 470mm in concrete. The base plates were 620mm x 320mm x 20mm (Figure 9).

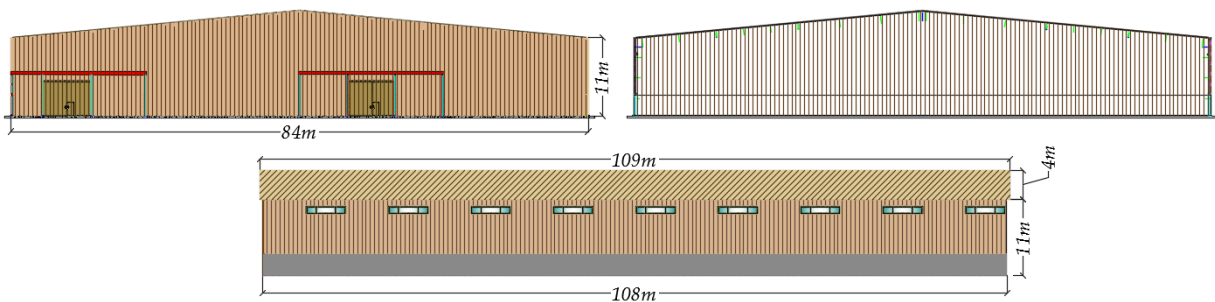


FIGURE 6. Front, Rear, and Side Elevations

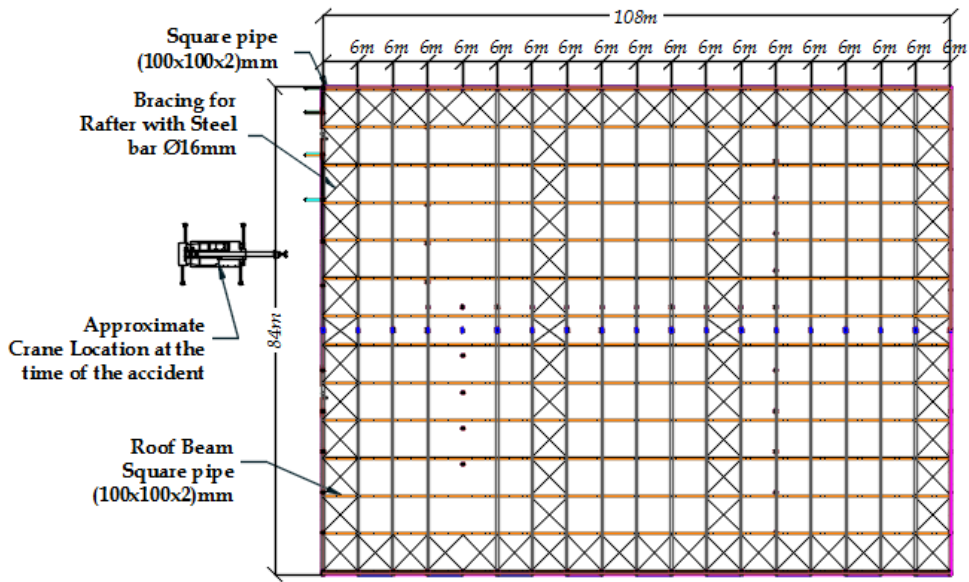


FIGURE 7. Portal Frame Structural Roof Plan

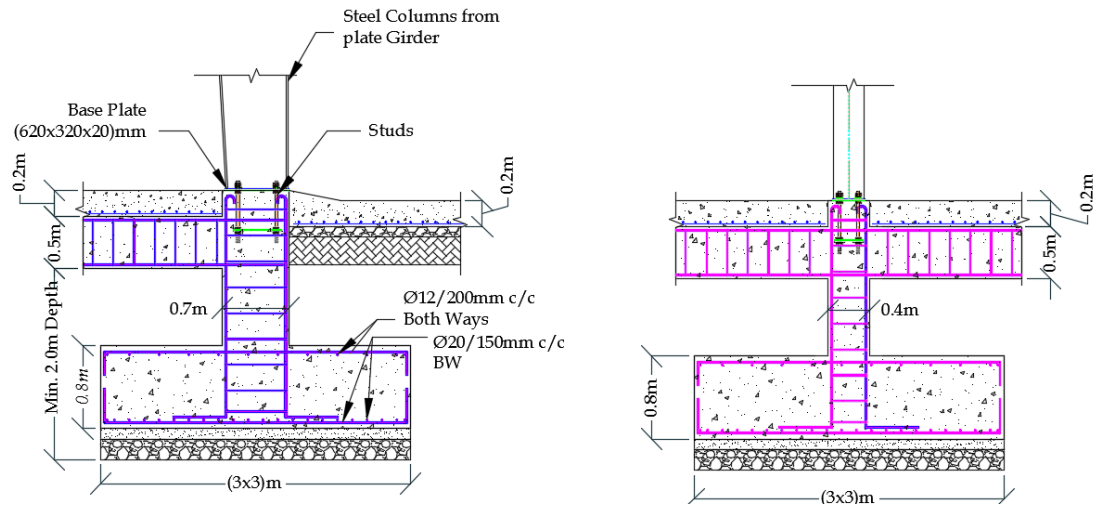


FIGURE 8. Footing Details

5. DESIGN LOAD

The frame has been designed for the following loads (Table 1):

- a) Self-Weight : Steel frame own weight
- b) Cladding and Purlin Load : 25 Kg/m²
- c) Service Load : 10 Kg/m²
- d) Snow Load : 30 Kg/m²
- e) Wind Loading of 120km/h as per ASCE7-10.

Seismic load is ignored since it seldom controls with this type of structure and in that region. A total of 34 Load combinations are created using LRFD method in accordance with ASCE7-10 (Table 2).

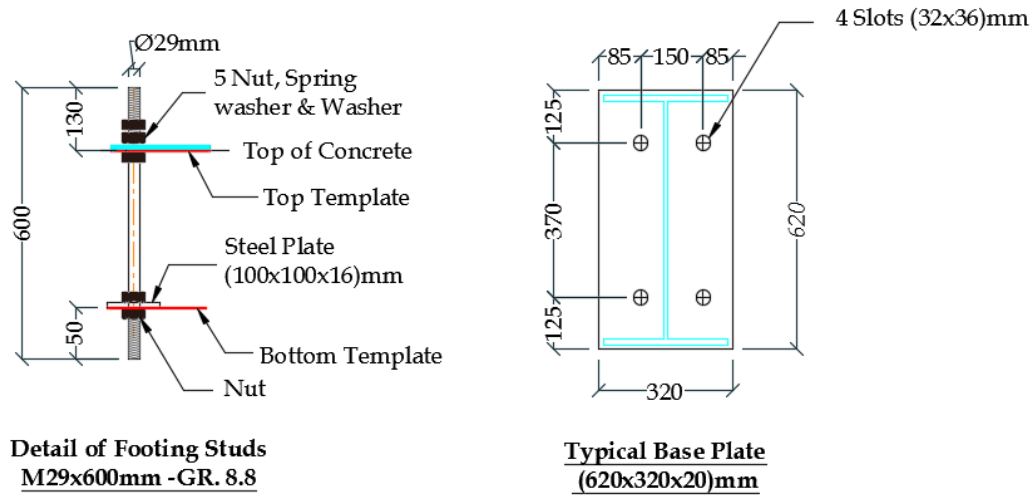


FIGURE 9. Anchor Rod and Base Plate Details

TABLE 1.
Design Loads

	Label	Case name	Nature	Analysis type
1	Self-weight	Self-weight	dead	Static - Linear
2	W_lr(+)	Wind L/R pres.(+)	wind	Static - Linear
3	W_lr(+)_C(+)_	Wind L/R pres.(+) Cp 1	wind	Static - Linear
4	W_lr(-)	Wind L/R pres.(-)	wind	Static - Linear
5	W_lr(-)_C(+)_	Wind L/R pres.(-) Cp 1	wind	Static - Linear
6	W_rl(+)	Wind R/L pres.(+)	wind	Static - Linear
7	W_rl(+)_C(+)_	Wind R/L pres.(+) Cp 1	wind	Static - Linear
8	W_rl(-)	Wind R/L pres.(-)	wind	Static - Linear
	Label	Case name	Nature	Analysis type
9	W_rl(-)_C(+)_	Wind R/L pres.(-) Cp 1	wind	Static - Linear
10	W_fr(+)	Wind Fr./Rear pres.(+)	wind	Static - Linear
11	W_fr(+)_C(+)_	Wind Fr./Rear pres.(+) Cp 1	wind	Static - Linear
12	W_fr(-)	Wind Fr./Rear pres.(-)	wind	Static - Linear
13	W_fr(-)_C(+)_	Wind Fr./Rear pres.(-) Cp 1	wind	Static - Linear
14	Wrf(+)	Wind Rear/Fr. pres.(+)	wind	Static - Linear
15	Wrf(+)_C(+)_	Wind Rear/Fr. pres.(+) Cp 1	wind	Static - Linear
16	Wrf(-)	Wind Rear/Fr. pres.(-)	wind	Static - Linear
17	Wrf(-)_C(+)_	Wind Rear/Fr. pres.(-) Cp 1	wind	Static - Linear
306	SNOW1	Snow case I	snow	Static - Linear
307	SNOW1	Snow case I edge	snow	Static - Linear
308	SNOW2_lr	Snow case II l/r	snow	Static - Linear
309	SNOW2_lr	Snow case II l/r edge	snow	Static - Linear
310	SNOW2_rl	Snow case II r/l	snow	Static - Linear
311	SNOW2_rl	Snow case II r/l edge	snow	Static - Linear

TABLE 2.
Design Load Combinations

Combs	Definition	Combs	Definition	Combs	Definition	Combs	Definition
ULS/ 1	1(1.4)	ULS/ 10	1(1.2) + 10(1.6)	ULS/ 19	1(0.9) + 2(1.6)	ULS/ 28	1(0.9) + 11(1.6)
ULS/ 2	1(1.2) + 2(1.6)	ULS/ 11	1(1.2) + 11(1.6)	ULS/ 20	1(0.9) + 3(1.6)	ULS/ 29	1(0.9) + 12(1.6)
ULS/ 3	1(1.2) + 3(1.6)	ULS/ 12	1(1.2) + 12(1.6)	ULS/ 21	1(0.9) + 4(1.6)	ULS/ 30	1(0.9) + 13(1.6)
ULS/ 4	1(1.2) + 4(1.6)	ULS/ 13	1(1.2) + 13(1.6)	ULS/ 22	1(0.9) + 5(1.6)	ULS/ 31	1(0.9) + 14(1.6)
ULS/ 5	1(1.2) + 5(1.6)	ULS/ 14	1(1.2) + 14(1.6)	ULS/ 23	1(0.9) + 6(1.6)	ULS/ 32	1(0.9) + 15(1.6)
ULS/ 6	1(1.2) + 6(1.6)	ULS/ 15	1(1.2) + 15(1.6)	ULS/ 24	1(0.9) + 7(1.6)	ULS/ 33	1(0.9) + 16(1.6)
ULS/ 7	1(1.2) + 7(1.6)	ULS/ 16	1(1.2) + 16(1.6)	ULS/ 25	1(0.9) + 8(1.6)	ULS/ 34	1(0.9) + 17(1.6)
ULS/ 8	1(1.2) + 8(1.6)	ULS/ 17	1(1.2) + 17(1.6)	ULS/ 26	1(0.9) + 9(1.6)		
ULS/ 9	1(1.2) + 9(1.6)	ULS/ 18	1(0.9)	ULS/ 27	1(0.9) + 10(1.6)		

The other side of this investigation includes the construction management to determine whether all the members of the structure are assembled per the design drawings and the specifications, if any, are followed. A field trip investigation was performed on March 18th 2013 for this purpose.

6. THE STRUCTURAL ANALYSIS RESULTS

As mentioned above, the 108m x 84m steel structure was modeled and analyzed in Autodesk Robot Structural Analysis Professional. The loads are applied as shown in the previous section. The structural analysis resulted in the following:

Outer Columns: The outer columns are tapered I sections. They all maintain their structural integrity for the loads with an interaction ratio of 0.26.

Center Columns: Center columns fail to resist the loads. Despite that it has an interaction ratio of 0.26, it fails the stability requirement with kL/r to exceed the maximum required of 200 by 16.64. These columns must be tied and braced to each other to reduce the unbraced length. The controlling column fails under the load combination number 16, which is dead load plus wind load.

Rafters: Rafters fail to resist the loads. The controlling member has an interaction ratio of 3.24. The controlling load combination is combination number 8.

Purlins: The purlins fail under the wind and snow load combinations. The controlling member got an interaction ratio of 3.24. This member also fails the stability requirement as mentioned above by a large amount. **Side Wall Bracing:** The controlling member, which did not have a sufficient section modulus, failed the stability requirement by approximately 200. The enveloping load combination is number 4. **Roof Bracing:** These members failed in both of the strength requirements. The controlling member has an interaction ratio of 57.26.

Base Plate Analysis: Base plates are analyzed using AISC ASD9 methodology. For that, the load combinations in the Autodesk Robot Structural Analysis Professional are changed to ASD. The results of the reaction forces are calculated and the envelop values are taken as conservative approach to evaluate the base plates. These envelop values are shown in Table 3 and Figure 10.

TABLE 3.
Envelop Reactions Per ASD Combinations

	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	31.24	0.99	102.45	0.32	81.61	0.01
Node	96	3	1254	96	96	101
Case	6	1	1	1	6	14
MIN	-31.18	-0.88	-110.88	-0.28	-81.61	-0.01
Node	94	101	1255	1176	94	99
Case	2	1	10	1	2	14

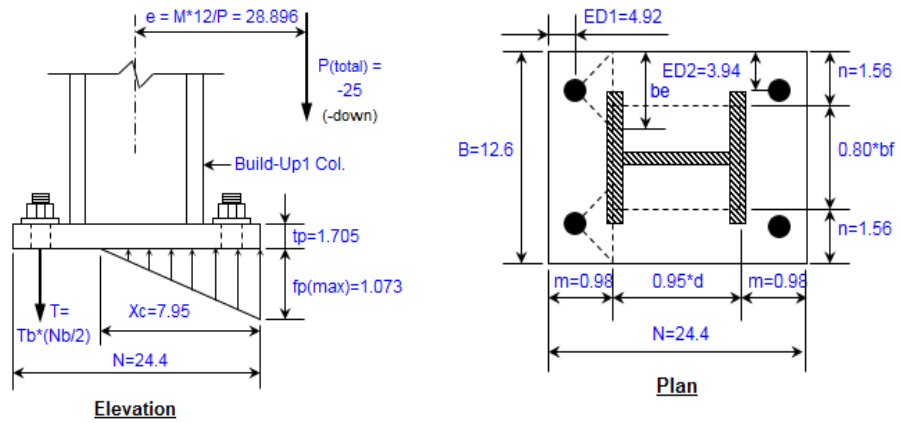


FIGURE 10. Base Plate Analysis

These values are inserted in an Excel sheet created specifically to analyze the base plates. The results show that the base plates are capable to resist the loads and maintain their structural integrity as follows. Note that the bearing pressure is exceeded the allowable by approximately 2%, which is acceptable in this case.

Front and Back Wall Bracing: The 4mm thick plate that was designed and used as bracing member, is not sufficient to handle the loads. The interaction ratio is huge.

Anchor Rod Analysis: The controlling footing was chosen for the analysis of the anchor rods since these bolts pulled out from the concrete. The base plate dimensions are 620mm x 320mm x 20mm and the anchor rods are $\phi 29$ mm M8.8 bolts of 600mm long. The base plates were supported by a concrete pedestal of 700mm x 400mm. As such, the base plate is flush oriented with the pedestal on the outer edge as shown in Section C-C of sheet number 32 of the design drawings. The anchor rods are embedded in concrete for a distance of approximately 470mm.

The representative core tests taken from the foundation on April 7, 2013 shows that the concrete passed the required compressive strength of 32 MPa. The results of the anchor rod analysis indicate that if they were installed as per the design drawings they were maintaining their structural integrity (Table 4). However, it is quite clear that the anchor rods failed to fulfill the requirements of the ASTM A307-97 since the elongation did not reach the lower limit [7]. According to ASTM A307-97, the minimum elongation percent in 50mm bar shall be 23% but the bars tested have elongations of 2.6%, 1.7%, and 1.4% as shown in Table 5. These values are too low. Based on the test results, although the tensile strength of the anchor rods has reached the required value, its elongation ratio did not meet the requirements. This has caused an abrupt pull out and/or premature ductile failure of the anchor rods which led the structure to collapse (Figure 11).

TABLE 4.
Summary Anchor Rod Analysis

Failure Type	Utilization Ratio
Steel Failure (T)	0.063
Concrete Breakout (T)	0.49
Pullout (T)	0.111
Side-face Blowout (T)	0.416
Steel Failure (V)	0.028
Concrete Breakout (V)	0.351
Concrete Pryout (V)	0.102
Stress Ratio	0.841

A trial and error was conducted to determine the lowest compressive strength of concrete that provides the sufficient capacity. The minimum f'_c that maintains the structural integrity and ensures that the anchor rods will not pull out from the concrete and the concrete will not crush under the interaction of shear and tension is 17.0MPa. The core tests show that the actual compressive strength exceeds the required value of 32MPa, which is almost twice as this value that ensures that concrete will not crush during stress. The high value of concrete compressive strength would have been an advantage if the anchor rods acted as ductile material but that never happened.

Therefore, the concrete crush is not from any glitches or quality from the foundation concrete but rather from the large impact pulling load that transferred almost wholly to concrete because the anchor rods failed to elongate as a ductile material.

TABLE 5.
Anchor Rod Lab Test Result

حدود المواصفات / الصنف Requirment(Grade)			Tensile Requirement
Grade A	Grade B	Grade C	
—	—	905 908 927	مقاومة الشد حد ادنى (Mpa)/N/mm2 Tensile Strength Min.
—	—	X2.6 X1.7 X1.4	الامتدادية لطول القياس 50 ملم حد الادنى % Elongation in (50) mm Min. % قطر القشبان (27.0 mm)



FIGURE 11. Anchor Rod Pull-Out Fail

7. ANALYSIS OF THE CONSTRUCTION ASPECT

It was noticed during the field trip that the rafter-column connection bolts were not snug tight and lacked the appropriate torque. This was noticed in several connections (Figure 12). This loose connection caused bolt thread failure in shear in some of these connection and bolt threads totally or partially sheared away as the result of the large dynamic impact from the collapsed members (Figure 13).



FIGURE 12. Snug Tight Connection During Construction



FIGURE 13. Shear Failure of Bolt Threa

As mentioned above in The Structural Analysis section, few anchor rods at the frame sections close to the assembly crane failed and caused the collapse of the frame sections. Few of these anchor rod failures were ductile as shown in Figure 14 (left) while several others pulled out from the concrete as shown in Figure 14 (right). This can be caused by a glitch in the design of the anchor rods that caused the concrete to crush under the pull-out force (Figure 14 and Figure 15) or by construction negligence. However, as discussed in the design section, the laboratory tests showed that the bolt materials lacked elongation; as such, did not act as a ductile material and caused this pullout failure. Few base plates failed besides the anchor rods. Some of them bent, while some others went one step further by failing under tear through by the anchor rods (Figure 16).

The base plate design evaluation shows that they have been designed to maintain their structural integrity for the applied loads. This indicates that the anchor rods were not tightened as required that permitted a dynamic impact on the anchor rod holes (Figure 16). On the other hand, the construction process of casing the pedestals that contained the anchor rods lacked quality work and common practice.



FIGURE 14. Anchor Rod Failure, Ductile Failure (left) and Non-Ductile Failure (right)

The pedestals that contained the anchor bolts were fairly short and was much better to cast them monolithically with the tie beams. The ideal method to cast the pedestals monolithically with the other parts of the foundation and install the anchor rods is by leaving a hole through

the top leveling plate to allow the pedestals to be casted concrete monolithically with the other parts of the foundation (Figure 17).

Anchor bolts must be designed to be ductile. To do this, the designing pullout strength of the concrete failure cone (U_p) must be equal to the minimum specified tensile strength ($F_u A_t$) (Shipp & Haninger, 1963). Unfortunately, this was not the case in this structure.



FIGURE 15. Anchor Rod Pull-Out and Concrete Crush Failure



FIGURE 16. Base Plate Tear Through Failure

Despite having small height, it was noticed that the foundation pedestals were not casted monolithically with the rest of the foundation but rather after casting the footing. This can be noticed from Figure 18 that a gap was left underneath the top plate that was filled with concrete after casting a portion of the pedestal. This created a construction joint not so far from the top plate and the contractor was not able to vibrate that portion of concrete and made it prone to having voids in the pedestals.



FIGURE 17. Typical Common Practice Pedestal Construction



FIGURE 18. Actual Pedestal Casting

In few column bases, it was noticed as shown in Figure 19 that the gap underneath the column base plate was filled when the floor slab was cast and therefore, created a weak joint between the old concrete of the pedestal that connects to the tie beams and that portion of it that is above it. This is also an indication that normal common practice was not followed and the construction process lacked quality. In addition, the construction management was poor. The bolts should have been tested in a certified laboratory and the designers' approval should have been taken prior to using them. Also, the manufacturer's certification is missing for the rest of materials used such as the nuts, washers, etc. As such, this flaw is a glitch in the construction management of the project that should become lesson-learned for future similar projects.



FIGURE 19. Gap Under Base Plates Casted with Floor

7. CONCLUSION

a. CONSTRUCTION MANAGEMENT AND MATERIALS

The following results are concluded based on the investigation:

I. The 108m x 84m portal frame structure was not erected in a safe way. Safety rules are not followed and wise engineering judgment was altered. The rigging process was not conducted in an appropriate way by continuing construction during an extreme wind that caused the crane to collapse.

II. Connection bolts were not snug tightened to ease the construction process. This method is very risky and is normally used when the connection holes are not in accurate spots so the installer does not tight the bolts until all the frame parts are installed. This is was a part of the collapse cause. The dynamic force associated with the collapse of the crane and the wind transferred the impact into the bolts. Since the bolts were not tightened enough, it caused a severe damage to the connection bolts that the threads of some of them sheared away by the impact. Also, few base plates failed under the impact load since the anchor rods were not snug tightened that caused a punch through the bolt holes in addition to severe buckling.

III. The catastrophic collapse of the structure occurred because of failure in the anchor rods and the base plates in addition to concrete failure that caused the anchor pull-out in few places.

IV. The abrupt-action collapse occurred as a result of bad anchoring the structure and bad connecting the rafters to the columns. Construction sequence of the member installation was managed poorly.

- V. The concrete of the footing pedestals where the columns are anchored to the footing crushed during the collapse. This must not happen under any circumstances because the anchor rods are designed correctly. The analysis of these anchor rods shows that they are designed to be ductile. This means if any failure has to happen, the anchor rods to fail prior to failure in concrete and prevent crushing concrete to avoid an abrupt collapse. As the concrete of the pedestals crushed in few places and the anchor rods pulled out from concrete, and the core tests show that the compressive strength of concrete meets the requirement of 32MPa, fingers are pointed to the material of the anchor rods. The anchor rods failed the elongation requirement test per ASTM A307-97 as per Seko Engineering House's report number 2468 on March 27, 2013. This means that the anchor rods did not act as ductile material and allowed concrete to crush.
- VI. The gap underneath the column base plate was filled when the floor slab was cast and therefore, created a weak joint between the old concrete of the pedestal that connects to the tie beams and that portion of it that is above it. As such, the weak spot in the anchor embedment was participated in the concrete breakout mechanism as per part iii of Figure RD.4.1 of ACI Code 318 [6].

b. **DESIGN**

As mentioned above in this paper, the warehouse structure was evaluated for structural integrity. The following are the conclusion of this analysis:

- I. The outer columns maintain their structural integrity per the design.
- II. The interior columns fail to maintain their structural integrity and lack lateral bracing.
- III. The rafters fail to maintain their structural integrity and are not qualified for the load.
- IV. None of the lateral bracings of the wall and the roof are capable to maintain their structural integrity for the load.
- V. The base plates are capable to maintain their structural integrity and are qualified for the load per the design.
- VI. The design anchor rods are capable to maintain their structural integrity and are qualified for the load per the design.

c. V3. CAUSE OF THE CATASTROPHIC COLLAPSE

Based on the evaluation shown above and calculations, fingers are pointed to both of the design and construction management. Since the design lacks integrity, constructing it is not safe either. However, it is believed that the catastrophic collapse is caused by construction mistakes, safety negligence, and not following the standards by not tightening the bolts and poorly managing the sequence of the construction process. Despite that the design lacks integrity; if it was constructed the right way then it would not be going to collapse under the circumstances of the time of failure because the wind that blew at that time did not have the sufficient pressure on the structure to push it down. This is because the structure was open in all directions and the siding was not installed yet. In this case, the structure can withstand its own weight plus a minor pressure from the wind. As such, the cause of the collapse is from the negligence in construction management and materials not meeting the design requirements. Details of the mistakes are outlined in Construction Management and Materials section clarified above.

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