

# Structural Investigation of the January 20, 2014 Plant Collapse at International Nutrition Facility in Omaha, NE

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U.S. Department of Labor  
Occupational Safety and Health Administration  
Directorate of Construction

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# **Structural Investigation of the January 20, 2014 Plant Collapse at International Nutrition Facility in Omaha, NE**

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## **Introduction**

On Monday, January 20, 2014 at about 9:40 a.m. a massive collapse occurred at a plant in Omaha, Nebraska producing nutritional supplements for animal feed. Two employees were killed and thirteen others were injured. The facility was located at 4444 S. 76<sup>th</sup> Circle, Omaha, NE. It is owned by International Nutrition, Inc. of Omaha, NE. The offices of International Nutrition, Inc. are located at the adjoining property at 7706 I Plaza, Omaha, NE. Bodies of the dead employees were retrieved within two days. The injured employees were able to get back to work after a few days. The structural collapse was so massive that it rendered the entire plant inoperable, and it had to be shut down. The central area of the building containing nine bins over the roof of the building and their supporting structure collapsed in a northerly direction and could be seen resting on the failed structure at an angle. See Figs. 1 thru 3, below. The remaining structure was in an unstable and precarious condition, and, therefore, had to be immediately abandoned. No one was permitted in the facility except under strenuous conditions.



**Fig.1** Aerial view of collapse



**Fig. 2** Aerial view of collapse



**Fig. 3** Aerial view of collapse

The arduous task of investigating and determining the cause of the collapse began. Initially, it was surmised that the collapse occurred due to an explosion but this was soon ruled out by the experts from the OSHA Salt Lake Technical Center due to the lack of a debris field, and a source of the explosion. Mr. Lee Hathon and Jedd Hill from SLTC visited the site multiple times. The focus then shifted to a structural collapse. Immediately efforts were made to locate the original structural framing plan for the bins, their supporting structure, and the building. A partial framing plan for the building was located but the plans for the bins could not be found. Considerable efforts were made to locate the manufacturer and fabricator of the bins which were completed in around 1972. Neither the owner nor the City Hall had any records. This resulted in a total lack of information on how the bins were manufactured and how were they supported over the framed buildings. This lack of information impeded the structural investigation because it was considered unsafe to step into the collapsed portion of the building to obtain any information. Demolition began on or about March 22, 2014 in a controlled manner, and approximately six weeks after the demolition began, the structural pieces began to be retrieved from the massive debris. It took approximately 3½ months before a serious structural investigation could begin.

A few days after the collapse, the OSHA Regional Administrator for Region VII asked the Directorate of Construction (DOC) in OSHA's National Office for engineering assistance in investigating the incident which had attracted considerable media attention. A structural engineer visited the site multiple times to gather information, observe the collapse, perform field measurements, and discuss the failure with the participants. Talks were held with the owner of the facility to obtain documents, to learn about the activities at the plant, the methods of operation, the process of manufacturing nutritional supplements for the animal feeds, and to take photographs. Following the location of documents, and performing field measurements of the failed structure, we conducted structural analyses of the failed structure, and our report follows. This report would not have been possible without the tireless efforts of Mr. Scott Jacobson, the lead compliance officer, and Ms. Bonita Winingham, Director of the Omaha OSHA Area Office.

### **The facility**

The facility was originally constructed around 1972, and it was owned by Vitamin Premixers of Omaha, Inc. Then around 1983, SmithKline Beecham Animal Health Company purchased the facility. The facility was then acquired by U.S. Pharmaceutical Company, Pfizer, in 1994. Finally, the present owner, International Nutrition Inc. (INI) acquired the facility around 1997. During the ownership of SmithKline, new legs were installed by PMI Nebraska Inc. around 1986 with a new conveyor at the top. See Figs. 4 thru 7.





**Fig. 4** Original facility



**Fig. 5** Original facility



**Fig. 6** Facility after modification



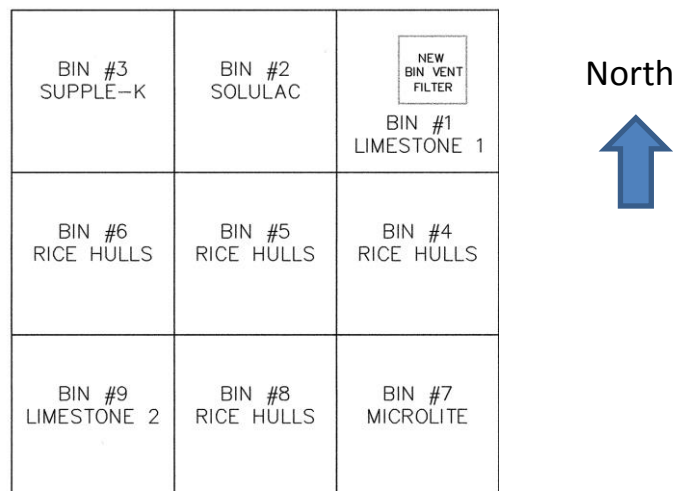
**Fig. 7** Two outside bins being added

When INI acquired the property, it engaged Ken Bratney Company (KBC) of Des Moines, IA for major modification of the facility. The following were the major changes made:

1. Adding two new mixers.
2. New bagging machine was added.
3. Installing a new receiving pit and tunnel with a new enclosure.
4. New elevator legs and their supports added.
5. New drag conveyor at the top platform added.
6. New 10-outlet electric distributor added.
7. Adding three additional pneumatic lines.
8. Installing platforms at and above the electric distributor.

See Fig. 6 showing the new legs, new platforms, conveyor, etc. All such modifications were performed around 1997. It must be noted that no changes were made to the original nine bins and their supporting structure. INI also retained PMI Nebraska, LLC of Grand Islands, NE to perform a variety of maintenance work at the plant from 1997 to 2014. But none of this work involved any structural modifications or major repairs to the bins or their support structure. KBC produced a set of drawings showing equipment layout, but to our knowledge no structural drawings were produced. By adding several platforms, a new electric distributor and a drag conveyor, KBC added substantial loads to the bins without modifying the bins' supporting structure.

It is interesting to note that on one of the KBC drawings, F1.8 dated July 2, 1996, there is a sketch showing nine bins with limestone in two diagonally opposite bins, Microlite in one corner bin, supple-K in the fourth corner bin, with the rest of the bins having rice hulls (see Fig. 8, below).



**Fig. 8** Bin layout (Taken from drawing F1.8 by Ken Bratney Company, 1996)

We understand as per KBC that the drawing merely indicated what KBC believed to be the current practice at that time. Microlite and Supple-K generally weighs 75 and 85 pounds per cubic foot (pcf), respectively. Limestone weighs approximately 90-95 pcf, and Rice Hull is approximately 20 pcf.

In 2006-2007, INI contracted with PMI to put a roof over the delivery pit, and to install two new outside storage bins, later identified as bins No. 10 and 11, see Fig. 7. In the summer of 2011, to meet the demands of a client, INI was planning to store limestone in bins No. 10 and 11 but decided against this on the advice of PMI who stated in an email that the outside bins could not support the heavier loads of limestone. Then attention turned to the existing nine bins on the roof where limestone, to some degree, was already being placed in bins No. 1 and 9 through existing pneumatic pipes. INI asked PMI to explore whether limestone could be stored in other bins on the roof. PMI consulted with a local engineering consultant, Reznicek Engineering Inc., to determine if that could be accomplished. Mark Reznicek visited the site a couple of times to take some field measurements of the supporting structure. He stated that he asked for additional information from INI to complete his analysis but that additional information never came. Reznicek said that then he abandoned the project. After this, things became murky. INI believes that PMI advised them at least verbally that other corner bins on the roof could support limestone. There is no e-mail or any document from PMI in existence that could substantiate this claim.

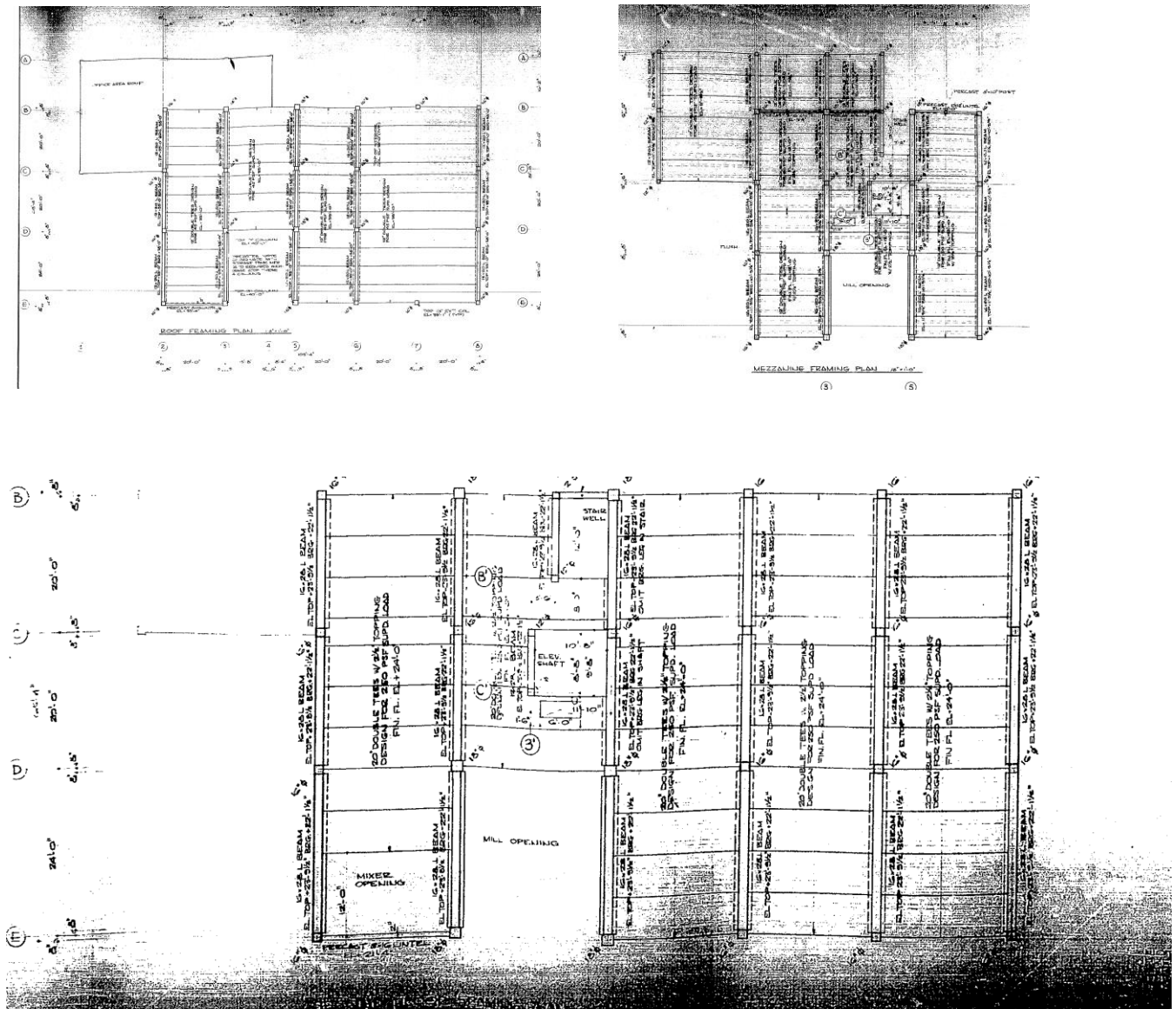
Limestone could only be transported to the bins through pneumatic pipes, and not through legs containing buckets because limestone is heavier. In 2012, PMI proposed to replace the existing pneumatic lines to bins No. 1 and 9, and install new pneumatic lines to bins 3 and 7 to deliver limestone in the bins. INI approved the proposal, and work was completed in the fall of 2012. It is not known whether PMI installed new pneumatic lines to bins No. 3 and 7 at the specific instructions of INI or PMI installed the lines because PMI believed that the bins 3 and 7 could safely support lime stone. In any event, PMI did not advise INI against installing pneumatic lines to two additional bins because a structural evaluation was not done. PMI installed the lines anyway, and INI managers stated that they believed that PMI must have determined that the bins could safely support limestone otherwise PMI would not have installed the pneumatic lines.

### **Existing facility:**

The facility consisted of a three story precast concrete structure with precast columns, inverted tee beams and double tees. There were three bays in the north south direction, and four bays in the east west direction. Typically, the bays were 20 ft. by 20 ft. except for the “mill opening” which was 24 ft. x 24 ft. The overhead bins were located over the mill opening. Generally the columns were 16”x16” except for the mill opening rows which were 18”x18”. All columns were precast, and were provided with corbels for the inverted tee beams to sit on. The inverted tee beams ran in the north-south direction. Double tees (12-20” deep) were provided in the east-west direction with 2 ½” topping on the second and third floors. Topping was not provided at



the roof level. Floor-to-floor height between the first, second and third floor was 12 ft. each. The third floor to roof height was 15 ft. There was no basement and the first floor was slab on grade. Fig. 9 indicates the original framing plans.



**Fig. 9** Original structural drawing of the concrete framings

### **The bins:**

All nine bins were part of the original structure believed to have been constructed in 1972-74. The bins were not pre-fabricated but were originally put together at the roof over the bin supporting structure. The 8'x8' square bins, 27' high, consisted of 1/8" steel plates welded

together. The intersecting walls of the bins were connected by welding the steel plates to corner steel angles, see Figs. 10 thru 13, below.



**Fig. 10** Bin construction



**Fig. 11** Bin construction



**Fig. 12** NE concrete column in bin No. 1



**Fig. 13** NE concrete column in bin No. 1

The walls of the bins were reinforced by welding 5x3x1/4" steel angles to the steel plate at 4' o.c., vertically. There was a common wall between the bins. The hoppers were welded to the bins, and were integrated to each other with a lip bearing over the top flange of the top chords of the external and internal trusses. Thus, the bins and the hoppers were one integrated structure resting over the steel beams, see Figs. 96 thru 98.

At the top of the bins were two roofs (horizontal diaphragms) consisting of 1/8" plates. There was a space of 6-8" between the two diaphragms. The lower diaphragm covered the top of the bins and was the roof of the bins. The upper diaphragm provided a platform for walking and maintenance work. See Figs. 14 & 15 below for the framing of the upper and lower platforms.



**Fig. 14** Roof bin



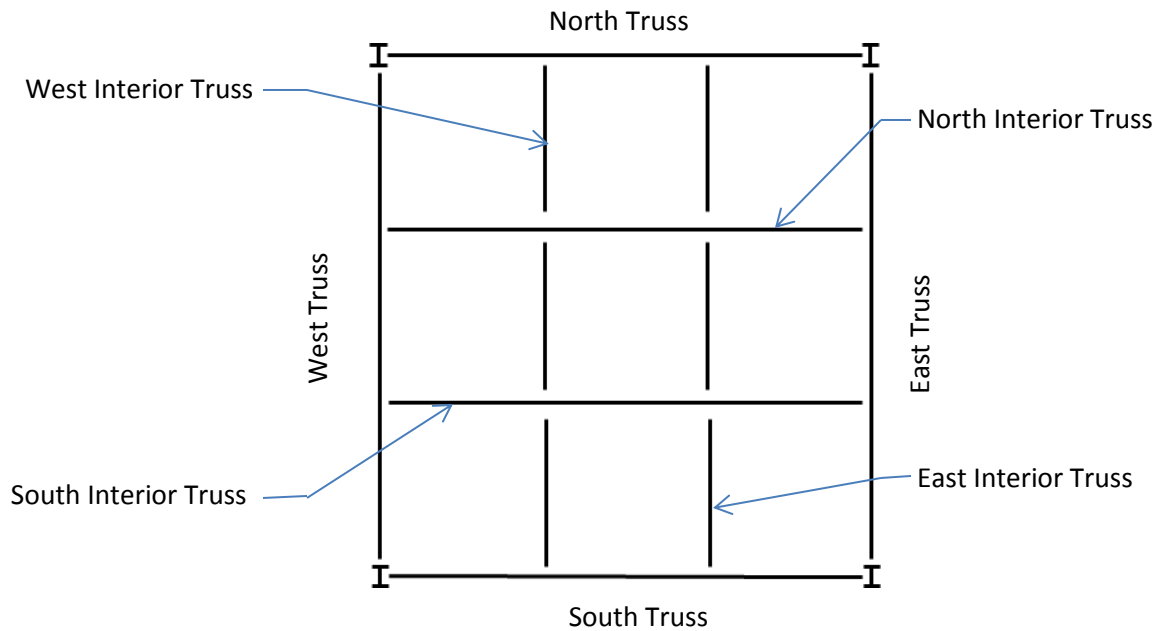
**Fig. 15** Roof bin

Around 1997, a major renovation was done to the facility when a new electric distributor and new platforms were added over the bins. Due to these renovations, an additional load of approximately 20,000 pounds were imposed on the bins (e.g., weight of the electric distributor, new platform, and new conveyor belts, etc.).

### **The bin supporting structure**

The structure supporting the bins and the hoppers rested on four corner steel columns, W8x40, approximately 10' high supported over 18x18" concrete columns identified as NE, NW, SE and SW columns. The steel column's base plates were welded to the steel plates embedded in these concrete columns. The columns were precast concrete reinforced with four #8 reinforcing bars with lateral ties placed at 18" o.c. In addition to supporting the bins, the concrete columns were part of the building frame supporting a roof, and the third and second floor framings. The bin supporting structure consisted of four exterior and four interior trusses, see Fig. 106 on next page.





**Fig. 106** Bin supporting structure

Two exterior trusses ran in an east-west direction and were directly supported over the steel columns resting on the concrete columns at the east and west ends of the bins. The other two exterior trusses ran in a north-south direction, and were framed to the steel columns through seated connections welded to the flange of the columns. Two interior trusses ran in the east-west direction framed to the east and west primary trusses. Similarly, two interior trusses ran in the north-south direction and were framed to the north and south primary trusses. The connections of the interior trusses to the exterior trusses were through the seated connections projecting from the top chord of the exterior trusses.

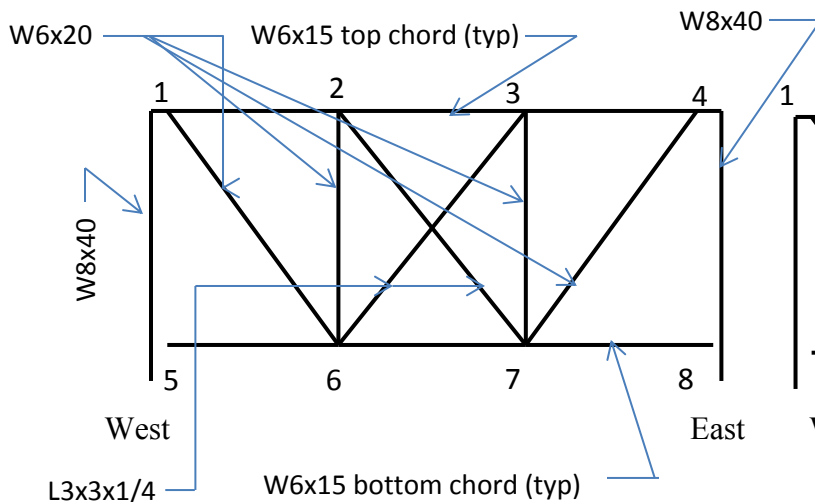
Unfortunately, original structural drawings of the framing prepared in around 1972 or any related fabrication drawings were not available, and so field measurements had to be done after the damaged trusses were retrieved during the demolition. The exterior trusses were 9 ft. deep whereas the interior ones were 6 ft. deep.

**Observation of the collapsed supporting structure:**

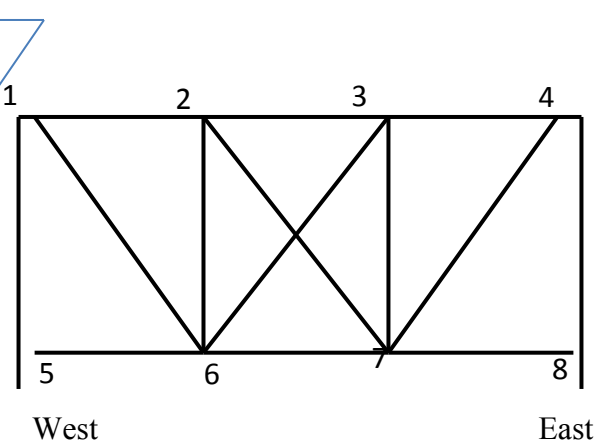
The top and bottom chords of the exterior and interior trusses were W6x15. Diagonals and vertical members were W6x20. The cross bracings were 3x3x1/4". The four supporting columns were W8x40 with a base plate which was field welded to an embedded steel plate in the concrete column. As stated earlier, the exterior and interior trusses were 9 ft. and 6 ft. deep,

respectively. The top chords of north and south trusses rested over W8x40 columns at each end, and were connected with four bolts. The reaction from the north and south trusses were transferred directly to the column centerline. The top chords were provided with web stiffeners. The east and west trusses were, however, framed to the columns differently. The top chords of the east and west trusses rested over the column cap plate which was extended some 6 inches beyond the flange of the column, and were connected by four bolts, see Fig. 93. This produced an eccentricity of approximately 7 inches to the center of the supporting column about its major axis. The ½" extended cap plate was provided with one 3/8" stiffener, as shown below. The top chords were also provided with web stiffeners.

All four beam-bearing connections on the east and west exterior trusses failed when the 3/8" stiffener plate buckled. The four connecting bolts remained mostly intact, see Figs. 34, 39 and 44. The north and south trusses are shown in Figs. 86 and 87 respectively. Their joints are shown in Figs. 16 thru 33 below.



**Fig. 86 North Truss**



**Fig. 87 South Truss**



**Fig. 16 – Joint 1 – North truss**



**Fig. 17 – Joint 2 – North truss**



**Fig. 18 – Joint 3 – North truss**



**Fig. 19 – Joint 4 – North truss**



**Fig. 20 – Joint 5 – North truss**



**Fig. 21 – Joint 6 – North truss**



**Fig. 22 – Joint 7 – North truss**



**Fig. 23 – Joint 8 – North truss**



**Fig. 24 – Joint 1 – South truss**



**Fig. 25 – Joint 1 – South truss**



**Fig. 26 – Joint 2 – South truss**



**Fig. 27 – Joint 3 – South truss**



**Fig. 28 – Joint 4 – South truss**



**Fig. 29 – Joint 4 – South truss**



**Fig. 30 – Joint 5 – South truss**





**Fig. 31** – Joint 6 – South truss

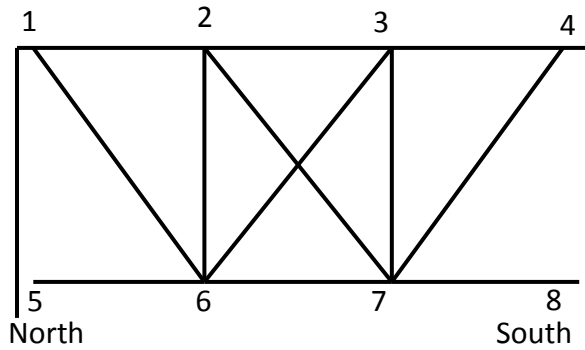


**Fig. 32** – Joint 7 – South truss

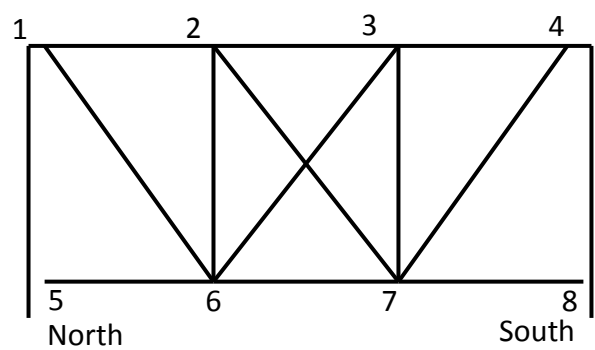


**Fig. 33** – Joint 8 – South truss

The east and west trusses are shown in Figs. 88 and 89 respectively. Their joints are shown in Figs. 34 thru 51 below.



**Fig. 88** East Truss



**Fig. 89** West Truss



**Fig. 34**– Joint 1 – East truss



**Fig. 35**– Joint 1 – East truss



**Fig. 36**– Joint 2 – East truss



**Fig. 37**– Joint 3 – East truss



**Fig. 38**– Joint 4 – East truss



**Fig. 39** – Joint 4 – East truss



**Fig. 40**– Joint 5 – East truss



**Fig. 41**– Joint 6 – East truss



**Fig. 42**– Joint 7 – East truss



**Fig. 43** – Joint 8 – East truss



**Fig. 44** – Joint 1 – West truss



**Fig. 45** – Joint 2 – West truss



**Fig. 46** - Joint 3 – West truss



**Fig. 47** – Joint 4 – West truss



**Fig. 48** – Joint 5 – West truss



**Fig. 49** – Joint 6 – West truss



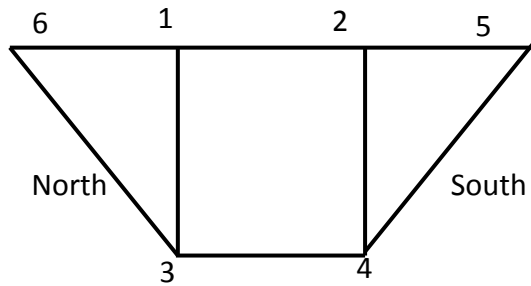
**Fig. 50** – Joint 7 – West truss



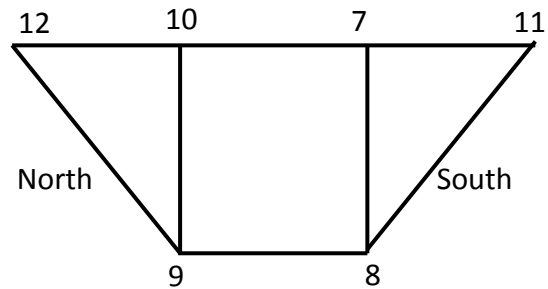
**Fig. 51** – Joint 8 – West truss



The east and west interior trusses are shown in Figs. 108 and 109 respectively. Their joints are shown in Figs. 52 thru 63 below.



**Fig. 108**– East Interior Truss



**Fig. 109**– West Interior Truss



**Fig. 52** – Joint 1 – East interior



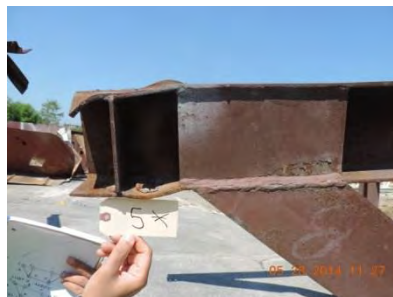
**Fig. 53** – Joint 2 – East interior



**Fig. 54** – Joint 3 – East interior



**Fig. 55** – Joint 4 – East interior



**Fig. 56** – Joint 5 – East interior



**Fig. 57** – Joint 6 – East interior



**Fig. 58** – Joint 7 – West interior



**Fig. 59** – Joint 8 – West interior



**Fig. 60** – Joint 9 – West interior



**Fig. 61** – Joint 10 – West interior

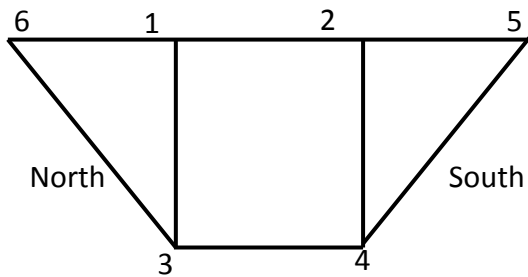


**Fig. 62** – Joint 11 – West interior

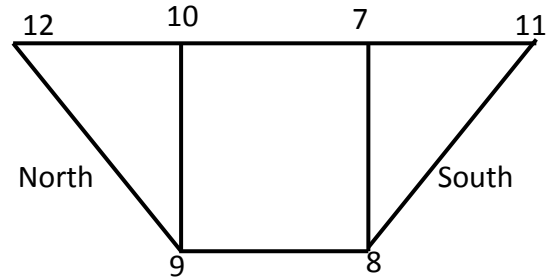


**Fig. 63** – Joint 12 – West interior

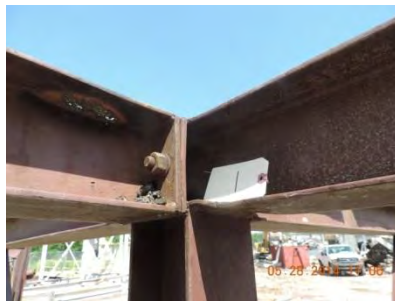
The north and south interior trusses are shown in Figs. 110 and 111 respectively. Their joints are shown in Figs. 64 thru 77 below.



**Fig. 110** – North Interior Truss



**Fig. 111** – South Interior Truss



**Fig. 64** – Joint 1 – North interior



**Fig. 65** – Joint 2 – North interior



**Fig. 66** – Joint 3 – North interior



**Fig. 67** – Joint 4 – North interior



**Fig. 68** – Joint 5 – North interior



**Fig. 69** – Joint 6 – North interior





**Fig. 70 – Joint 7 – South interior**



**Fig. 71 – Joint 7 – South interior**



**Fig. 72 – Joint 8 – South interior**



**Fig. 73 – Joint 8 – South interior**



**Fig. 74 – Joint 9 – South interior**



**Fig. 75 – Joint 10 – South interior**



**Fig. 76 – Joint 11 – South interior**



**Fig. 77 – Joint 12 – South interior**

The four seated connections of the north and south trusses over the column cap plate did not exhibit any signs of failure except the damage sustained during the collapse. The top chords of all exterior trusses did not exhibit severe buckling except at some panel points, although it cannot be determined whether the buckling occurred pre- or post-collapse. The bottom chords of all the exterior trusses contained severe distortions; see Figs. 21, 22, 32, 41, 42, 49, 50, and 107 believed to have occurred during collapse. The connection plates at the bottom chord of all the exterior trusses to W8x40 remained intact, and the bottom chords near the connections remained relatively undamaged.

The four interior trusses escaped serious damage and emerged relatively intact from the debris see Figs. 104 and 105. The top chords of the north and south interior trusses were continuous

and were connected to the east and west exterior trusses by four bolts on an extended gusset plate. The top chords of the east and west interior trusses were simple spans and were connected by two bolts to the north and south interior trusses. It is noteworthy that the bottom panel joints of all four interior trusses were practically undamaged; see Figs. 54, 55, 59, 60, 66, 67, 74 and 75.

The four corner columns, W8x40, remained intact and did not exhibit any signs of distress. The failure occurred at the bottom of the columns where the welds to the column base plate failed, see Figs. 78 thru 85. The column base plate remained connected to the embed plates of the concrete columns; see Figs. 78 thru 85 and 95. For typical hoppers pictures after they have been retrieved from the debris, see Figs. 96 thru 98.



**Fig. 78 – W8x40 base plate**



**Fig. 79 – W8x40 base plate**



**Fig. 80 – W8x40 base plate**



**Fig. 81 – W8x40 base plate**





**Fig. 82** – W8x40 base plate



**Fig. 83** – W8x40 base plate



**Fig. 84** – W8x40 base plate



**Fig. 85** – W8x40 base plate

During the collapse, all four concrete columns remained intact except the NE column which was sheared at approximately the level of the inverted tee beam at the third floor, and was found inside bin No.1 during the demolition and recovery. The reason of the shear at that location was the lack of adequate lap splice of the longitudinal bars on one face of the column, see Figs. 99-103.

### **Structural analyses**

The most significant factor in evaluating bin structure is to determine the ratio of the loads carried by the bins, and by the underlying supporting structure. This factor depends upon the angle of the internal friction of the granular material, the coefficient of friction between the material and the bin walls and the ratio of the area to the perimeter of horizontal cross section of the bins. For this analysis we have taken a factor of 30% indicating that 70% of the gravity loads of the material stored in the bins would be directly transferred through the walls of the bins to the four supporting columns (W8x40) directly, and only 30% of the loads would be transferred

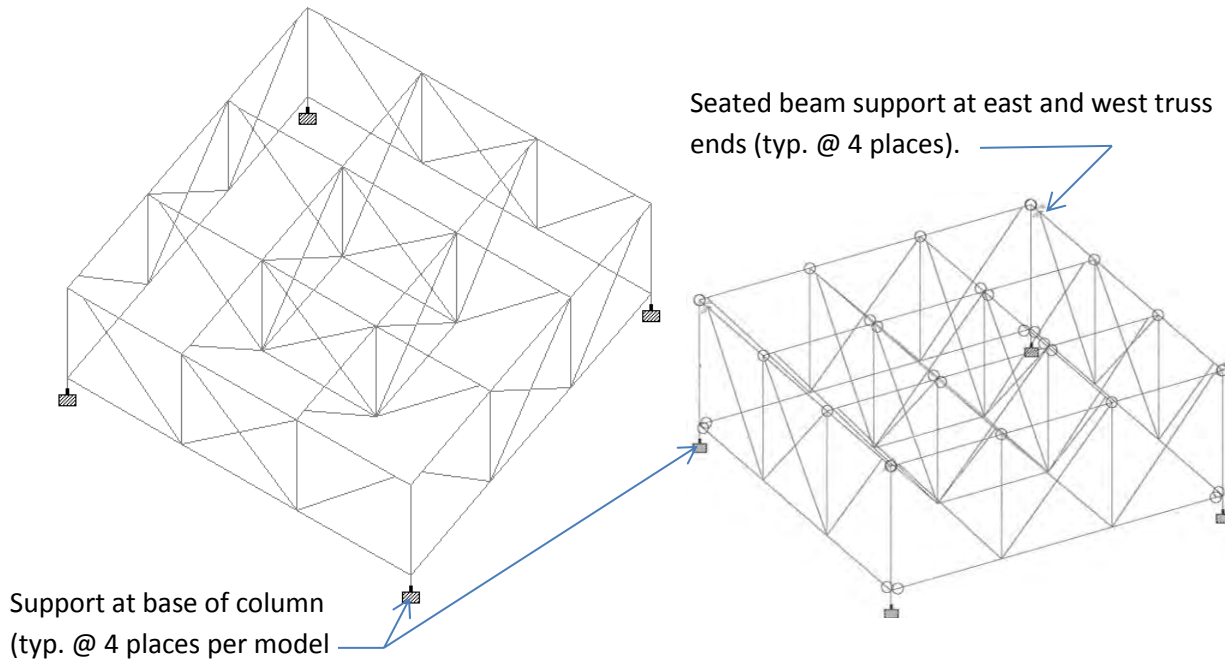
through the bottom hopper to the trusses. The distribution between the frictional force and the net vertical force was estimated using the Janssen's method which is well recognized by the industry for the design of bins for storing granular materials.

The supporting structure of the bins consisted of two exterior and two interior trusses spanning in a north-south direction, and two exterior and two interior trusses in the east-west direction. Exterior trusses were supported on four steel columns which were connected to the four concrete columns, identified as NE, NW, SE and SW columns. Framing for all four exterior trusses were identical, see Figs. 86-89, except for the end connections. The framing for four interior trusses was also identical. The connections of the four corner steel columns to the concrete columns were also identical.

As the facility is believed to have been constructed during 1972-1974, the grade of steel for all wide flange shapes and plates was assumed to be ASTM A-36. The sizes of the top and bottom chords, diagonals, verticals, and the stiffened beam connections were field-measured, as shown in Fig. 86. The top and bottom chords were W6x15. Diagonals and verticals were W6x20. The four primary columns over the top of concrete columns were W8x40. The steel bins were constructed with 1/8" steel plates welded together with stiffeners and corner angles. The bins had two layers of roof plates 6" apart, also consisting of 1/8" plates. It was estimated that an additional load of 20,000 pounds consisting of distributor, platforms, drag conveyors, etc. were imposed on the bins, but were directly transferred to the four corner columns. The trusses were assumed not to share any load coming from the roof of the bins. It is noteworthy that the bins were many times stiffer than the trusses, and they would receive a much greater share of the loads than the trusses.

For the purpose of analysis, limestone was regarded as weighing 90 pcf, and the rice hulls as 20 pcf.

Two models were created to assess the stresses in different members of the trusses under different loading conditions. The first model provided the forces imposed on the stiffened seated connection by the east and west trusses at the north and south ends, and the second model provided the forces and stresses in the truss members. The first model more accurately reflected the existing framing conditions at the stiffened seat connection. Analyses were done under the Load Resistance and Factored Design (LRFD), and under Allowable Stress Design (ASD). Under the LRFD, no increase in loads was assumed by using a load factor of 1.0, and no capacity reduction was taken by using the phi factor as 1.0. The intent under the LRFD evaluation was to compute the failure load at which a collapse could occur. ASD evaluations provided the usual factors of safety in accord with the industry practice. A finite element analysis and hand computations were performed to compute the load that the stiffened seat connection could carry. Three dimensional views of the two models, and the finite element plate analysis are shown below, see Figs. 90 thru 94.

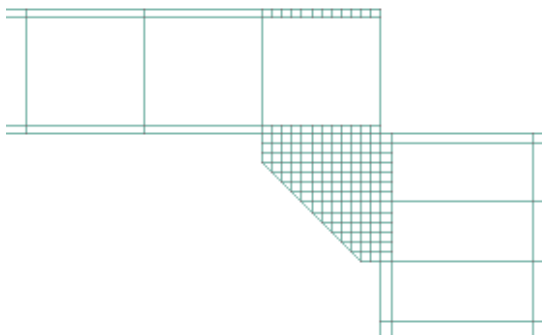


STAAD MODEL 2

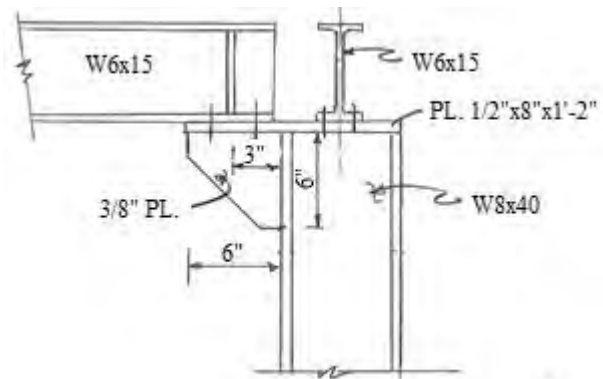
**Fig. 90**

STAAD MODEL 1

**Fig. 91**

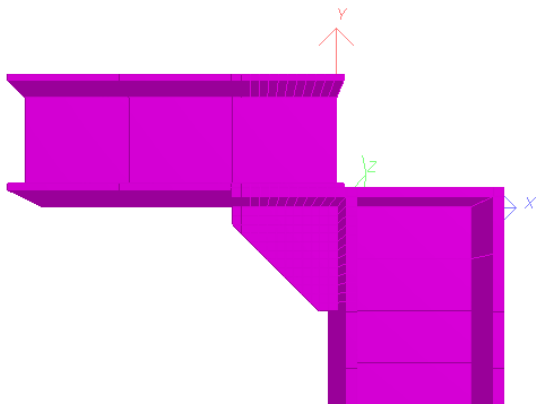


**Fig. 92** Elevation  
(Finite element model)



**Fig. 93** Elevation  
(Seated beam connection of the  
east and west truss)





**Fig. 94** Finite element model



**Fig. 95** – Intact column base plate



**Fig. 96** – Typical hopper



**Fig. 97**-Typical hopper



**Fig. 98** – Typical hopper



**Fig. 99** – NE corner column lying in bin No. 1



**Fig. 100** Fracture of NE corner column  
at 3<sup>rd</sup> floor



**Fig. 101** Another view of fracture of NE  
column at 3<sup>rd</sup> floor



**Fig. 102** NE corner column showing  
inadequate lap splice



**Fig. 103** NE corner column showing  
inadequate lap splice





**Fig. 104** Interior trusses lying in debris



**Fig. 105** Interior trusses being retrieved



**Fig. 107** Exterior truss after being retrieved



The analysis was conducted using the following criteria:

1. The steel conformed to ASTM A-36, as the bins were constructed around 1972 at which time high strength steel was just beginning to emerge into the market, although at a premium price, and was generally not used in conventional framing until the 1980s.
2. The structure was analyzed under the Load Resistance Factored Design (LRFD) code established by the American Institute of Steel Construction (AISC); except the load factor and the capacity reduction factors were assumed as 1.0 to eliminate any factor of safety.
3. The structure was also analyzed using Allowable Stress Method with the usual factors of safety.
4. No allowance for any corrosion was provided.
5. The bins were constructed with 1/8" thick steel plate, and reinforced with horizontal stiffeners of unequal 5x3x1/4" steel angles at 4'-0" on centers vertically.
6. The hopper (aka cone) was an integral part of the bins, and they were supported over steel beams on all four sides.
7. Regardless of the quantity of the material in the bins, the hoppers were assumed to be full.
8. Scales, mixers, and conveyor belts below the hoppers were supported independently of the bins and hoppers, and had their own supporting structure.
9. The truss members were field-measured and generally were comprised of W6x15 except for the diagonals and vertical members which were W6x20. The four corner steel columns conformed to W8x40. The cross bracings in the trusses were comprised of steel angles 3x3x1/4".
10. Rice hulls weighed 20 pounds per cubic foot (pcf).
11. There were two types of limestone powder stored in the bins, PureCal from Cerne Calcium Company of Fort Dodge, IA weighing 95 pcf, and Unical from Cilc Resources of Urbandale, IA weighing 84 to 92 pcf. For the analysis, 90pcf has been taken for all limestone.
12. Dry Distilled Grain (DDG), aka Solulac, weighed 20 pcf similar to the rice hulls.
13. Bin 9 was partitioned into two equal bins, but contained the same material in both parts but of varying quantities.
14. At the time of collapse, the loads in the bins were as follows:
  - Bins 1,3 and 7 combined approximately 375,000 pounds (Each bin assumed to carry approximately 125,000 pounds)
  - Bin 2: 33,000 pounds
  - Bin 4: 30,000 pounds
  - Bin 5: 28,000 pounds
  - Bin 6: 20,000 pounds
  - Bin 8: 26,000 pounds
  - Bin 9: 87,000 pounds

A three-dimensional frame analysis was conducted to determine the stresses in the bin supporting frame under varying load conditions. The following are the 24 different cases considered in the analyses.

1. Case 1: To determine whether the existing structure could support limestone in the four corner bins, and rice hulls in the remaining five bins under LRFD method (no factor of safety) using model No.1. All bins were assumed to be filled up to 85% of the capacity. Absence of factor of safety is a violation of industry standards.  
**Result: All truss members passed but the seated beam connections failed.**
2. Case 2: Same as Case No.1 but using model No. 2. Again no factor of safety.  
**Result: All truss members passed but the seated connections failed**
3. Case 3: To determine whether the existing structure could safely support limestone in four corner bins, and rice hulls in the remaining five bins with the required factor of safety (using ASD method), as per industry standards, using model No.1. All bins were filled up to 85% of capacity.  
**Result: Truss members failed and connections also failed**
4. Case 4: Same as Case 3 but using model No. 2.  
**Result: Truss members failed. All connections also failed.**
5. Case 5: To determine whether the existing structure could support rice hulls in all nine bins under LRFD (no factor of safety) using model No.1. All bins were filled up to 85% of capacity.  
**Result: All members and connections passed.**
6. Case 6: Same as Case 5 but using model No. 2.  
**Result: All members and connections passed.**
7. Case 7: Same as Case 5 but with the required factor of safety (using ASD method) using model No.1  
**Result: All members and connections passed.**
8. Case 8: Same as Case 7 but using model No. 2.  
**Result: All members and connections passed.**
9. Case 9: To determine whether the existing structure could support the loads of the material believed to be in the bins at the time of collapse using LRFD method (without any required factor of safety) using model No. 1.  
**Result: All members passed but the connections failed.**
10. Case 10: Same as Case 9 but using model No. 2.  
**Result: All members passed but connections failed.**
11. Case 11: Same as Case 9 but with the required factor of safety (using ASD method) using model No. 1  
**Result: All members passed but connections failed.**
12. Case 12: Same as case 11 but using model No. 2  
**Result: All members passed but connections failed.**
13. Case 13: To determine whether the existing structure could support four corner bins filled with limestone up to 65% of the capacity with the rest of the five bins filled with rice

hulls up to 85% of the capacity using LRFD method (without any factor of safety) using model No. 1.

Result: All members passed but the connections failed.

14. Case 14: Same as Case 13 but using model No. 2

Result: All members passed but the connections failed.

15. Case 15: Same as Case 13 but with the required factor of safety using (using ASD method) model No. 1.

Result: All members passed but the connections failed

16. Case 16: Same as Case 15 but using model No. 2

Result: All members passed but the connections failed.

17. Case 17: To determine whether the existing structure could support limestone in two corner bins filled up to 85% capacity with the rest of the bin filled with rice hulls up to 85% capacity using LRFD method (without any factor of safety) using model No. 1

Result: All members passed, and the connections barely passed.

18. Case 18: Same as Case 17 but using model No. 2

Result: All members passed, and the connections barely passed.

19. Same as Case 17 but with the required factor of safety (using ASD method) using model No. 1

Result: All members pass but the connections fail.

20. Same as case 19 but using model No. 2.

Result: All members passed but the connections failed.

21. To determine whether the four corner bins could support lime stone filled up to 55% of the capacity with the rest of the bins filled with rice hulls up to 85% of capacity using LRFD method (without any factor of safety) using model No. 1.

Result: All members and connection passed.

22. Same as Case 21 but using model No. 2.

Result: All members and connections passed

23. To determine whether the existing structure could support lime stone in the four corner bins filled up to 20% of the capacity with the rest of the bins filled with rice hulls up to 85% of the capacity using ASD method (with the required factor of safety) using model No. 1.

Result: All members and connections passed.

24. Same as Case 23 but using model No. 2.

Result: All members and connections passed.

Above analyses' results are summarized in Table 1 on next page.

**TABLE 1**

<b>Summary of the analyses results of the bin supporting truss structure</b>											
No.	STAAD FILE No.	No. of supports	No. of corner bins with LS	Rest of Bins with RH	Load coefficient	% of height fill	LS density	RH density	ASD or LRFD method	Truss results	Seated beam connection
1	1	Model 1	4	5	0.3	85% LS 85% RH	90	20	LRFD	Pass	<b>Fails</b>
2	1A	Model 2	4	5	0.3	85% LS 85% RH	90	20	LRFD	Pass	<b>Fails</b>
3	3	Model 1	4	5	0.3	85% LS 85% RH	90	20	ASD	<b>Fails</b>	<b>Fails</b>
4	3A	Model 2	4	5	0.3	85% LS 85% RH	90	20	ASD	<b>Fails</b>	<b>Fails</b>
5	2	Model 1	0	9	0.3	85% RH	No limestone	20	LRFD	Pass	Pass
6	2A	Model 2	0	9	0.3	85% RH	No limestone	20	LRFD	Pass	Pass
7	4	Model 1	0	9	0.3	85% RH	No limestone	20	ASD	Pass	Pass
8	4A	Model 2	0	9	0.3	85% RH	No limestone	20	ASD	Pass	Pass
9	5	Model 1	4	5	0.3	Actual loads	90	20	LRFD	Pass	<b>Fails</b>
10	5C	Model 2	4	5	0.3	Actual loads	90	20	LRFD	Pass	<b>Fails</b>
11	5B	Model 1	4	5	0.3	Actual loads	90	20	ASD	Pass	<b>Fails</b>
12	5A	Model 2	4	5	0.3	Actual loads	90	20	ASD	Pass	<b>Fails</b>
13	6	Model 1	4	5	0.3	65% LS 85% RH	90	20	LRFD	Pass	<b>Fails</b>
14	6A	Model 2	4	5	0.3	65% LS 85% RH	90	20	LRFD	Pass	<b>Fails</b>
15	7	Model 1	4	5	0.3	65% LS 85% RH	90	20	ASD	Pass	<b>Fails</b>
16	7A	Model 2	4	5	0.3	65% LS 85% RH	90	20	ASD	Pass	<b>Fails</b>
17	8	Model 1	2 bins at NE & SW	7	0.3	85% LS 85% RH	90	20	LRFD	Pass	Barely passes
18	8A	Model 2	2 bins at NE & SW	7	0.3	85% LS 85% RH	90	20	LRFD	Pass	Barely passes
19	9	Model 1	2 bins at NE & SW	7	0.3	85% LS 85% RH	90	20	ASD	Pass	<b>Fails</b>
20	9A	Model 2	2 bins at NE & SW	7	0.3	85% LS 85% RH	90	20	ASD	Pass	<b>Fails</b>
21	10	Model 1	4	5	0.3	55% LS 85% RH	90	20	LRFD	Pass	Pass
22	10A	Model 2	4	5	0.3	55% LS 85% RH	90	20	LRFD	Pass	Pass
23	11	Model 1	4	5	0.3	20% LS 85% RH	90	20	ASD	Pass	Pass
24	11A	Model 2	4	5	0.3	20% LS 85% RH	90	20	ASD	Pass	Pass

Note: Model 1 is generated with 8 support condition to determine forces at the seated beam connections and to determine truss member forces. Model 2 is generated with 4 supports condition to compare forces in the truss members and seat beam connections with Model 1.

The weakest link in the structure was the four stiffened seated connections of the top chords of the east and west trusses at each end. The bottom flanges of the top chords of the east and west exterior trusses, W6x15, were bolted with four bolts to the ½” cap plate of the corner column projecting beyond the column flanges. The cap plate was welded to the column. A stiffener plate 3/8” thick was provided underneath the cap plate. The seated connection was subjected to a vertical load coming from the east and west trusses and a horizontal force equivalent to the compressive force in the truss top chords. Both the vertical and horizontal forces created eccentricity on the connection, and it was determined that a single stiffener plate would overstress the seated connection to failure. If either a north or south connection was provided with a sliding joint to relieve the horizontal force, the connection with only one stiffener plate would have been able to resist the loads satisfactorily. Alternatively, if two stiffener plates were provided as was done in the cases of the interior trusses, the stresses would be satisfactory. As stated earlier, all four stiffened seated connections failed, see Figs. 34, 35, 38, 44 and 47.

### **Conclusions:**

Based upon the above, we conclude that:

1. The cause of the collapse was the failure of the four seated connections of the east and west exterior trusses at their north and south ends under the loads of limestone placed in the corner bins and other products in the remaining bins at the time of the collapse. The four seated connections were provided with only one stiffener plate unlike other similar connections of the interior trusses which were provided with two plates. If either two stiffener plates were provided instead of the one, or if one end of the east and west trusses was placed on a sliding pad, the collapse would not have occurred in spite of the limestone in the corner bins, and other products in other bins.
2. Throughout the forty-plus years' history of the plant, there is no record available to establish that there ever was a structural evaluation of the bins' supporting structure to determine whether limestone could be placed either in the four corner bins or in two diagonally opposite corner bins.
3. The weakest link in the structure was the load bearing capacity of the stiffened seated connection mentioned in conclusion #1, and was the controlling factor in determining load capacities of the bins. If a structural evaluation was done, this weak link would have been discovered, and this incident would not have occurred.
4. International Nutrition, Inc. began to place limestone in the four corner bins without either receiving any document from the previous owners establishing the structural adequacy of the bins to support the weight of limestone or other heavier materials, or conducting an independent structural evaluation to verify the same. Therefore, Section 5(a)(1) of the OSH Act was violated.
5. The original intent of structural design of the bins' supporting structure appeared to store rice hulls (20 pcf) in the bins, and not limestone. The supporting steel structure can

support rice hulls up to the 100% capacity in all nine bins with adequate factors of safety, as per industry standards.

6. The structure could not support limestone (90 pcf) in the four corner bins and rice hulls (20 pcf) in the remaining five bins at 85% of the capacity, with or without any factors of safety. However, the four corner bins could support limestone up to a maximum of 55% of bin capacity with rice hulls in the remaining five bins filled to 85% capacity without any factor of safety, a violation of the industry standards.
7. The structure could support limestone in four corner bins filled up to 20% of the capacity with the remaining bins filled with rice hulls up to 85% of the capacity with the required factor of safety.
8. If the four corner bins were filled with limestone greater than 55% of the bin capacities with the remaining five bins with rice hulls filled up to 85% of capacity, the collapse of the structure would be imminent.
9. The structure could support limestone up to 85% of the bin capacity in two diagonally opposite corner bins with the remaining bins filled with rice hulls up to 85% of capacity, but without any factor of safety which is a violation of industry standards.