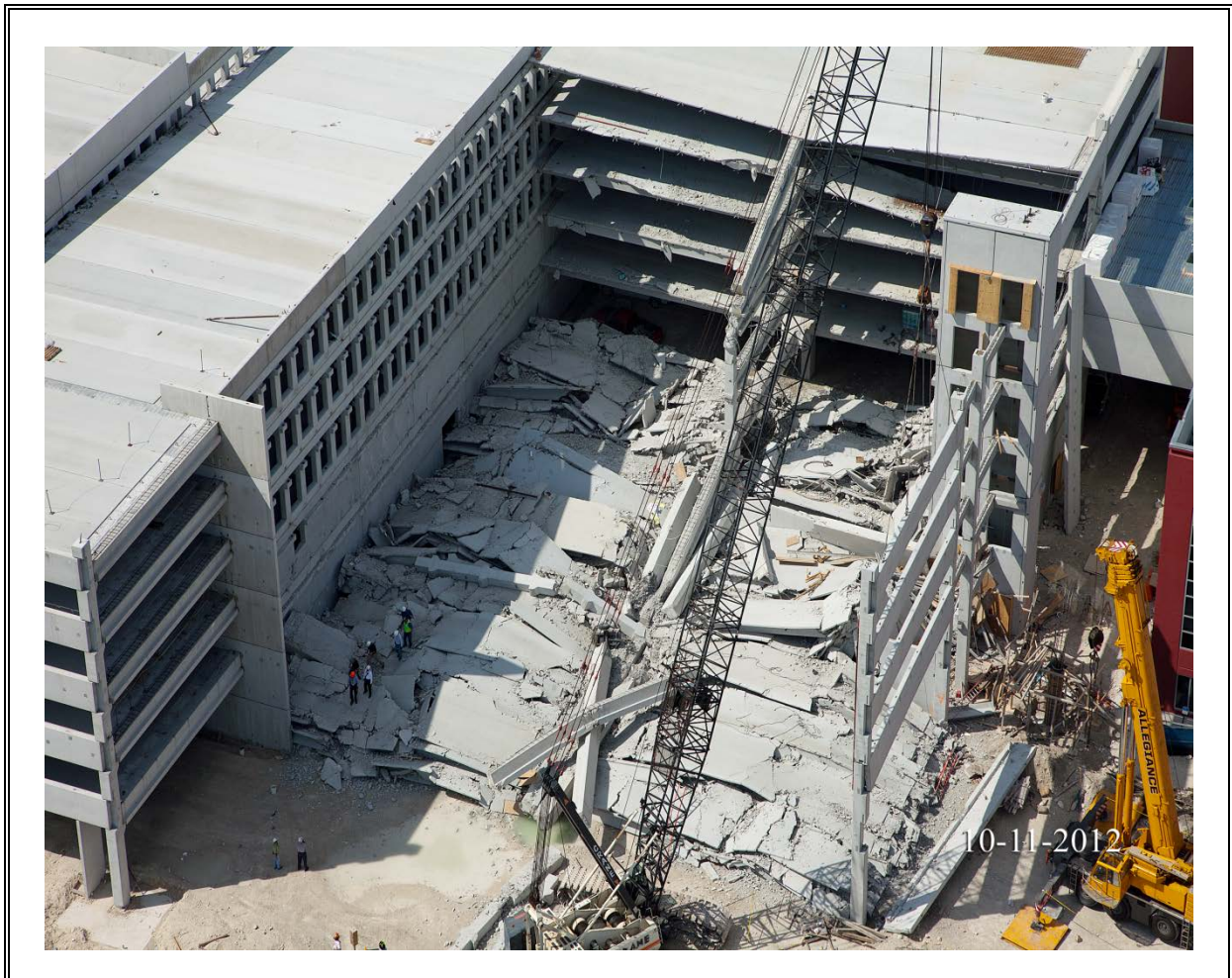


INVESTIGATION OF THE OCTOBER 10, 2012 PARKING GARAGE COLLAPSE DURING CONSTRUCTION AT MIAMI DADE COLLEGE, DORAL, FL

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

March 2013



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Background

On October 10, 2012 a construction catastrophe occurred in the Miami-Dade College West Campus at 3800 NW 115th Avenue, Doral, Florida. A six-story parking garage was under construction when the northeast portion of the garage suddenly collapsed, trapping and killing four employees and injuring three others. Fire and rescue personnel reported to the site immediately, and began the painstaking task of locating bodies under the massive rubble of fallen precast concrete structural elements from five floors, weighing approximately 3,300 tons. The collapse occurred over an area of approximately 122 ft. by 132 ft. The fourth victim was not retrieved until approximately a week after the collapse due to the arduous task of moving the heavy concrete pieces.

The Occupational Safety and Health Administration's (OSHA) Regional Administrator, Region IV, requested the Directorate of Construction (DOC), OSHA National Office in Washington, DC to provide technical assistance in a causal determination, and engineering assistance to the Ft. Lauderdale OSHA Area Office in its investigation. Two structural engineers from the DOC arrived at the incident site the day after the incident and remained there for two weeks. A few weeks later, another structural engineer from DOC visited the incident site and conducted interviews with the major participants in the project.

Construction documents, specifications, shop drawings and contractual papers were obtained from key contractors. Photos and videos were taken at the incident site and at the storage yard where the fallen major structural members were stored for examination.

DOC's investigation began immediately after the incident, and consisted of a review of all the construction documents, a structural analysis to determine whether the structure was designed properly in accord with the Florida Building Code, a review of the methods of construction, and conducting forensic engineering. The following is our report.

Introduction

The Board of Trustees of Miami Dade College (MDC) awarded a guaranteed Maximum Price Design-Build contract to Ajax Building Corporation (Ajax) of Florida, to design and construct a 1,800-car parking garage on the west campus of the college at 3800 NW 115th Avenue, Doral, FL. Ajax has multiple offices in Florida, and in Atlanta, GA. The project cost was approximately \$25 million. The garage was designed as a six-story precast concrete structure with precast columns, beams, double tees and wall panels. The beams and tees were pre-tensioned. There was also some cast-in place concrete. An existing warehouse was demolished to facilitate part of the construction.

The contract consisted of two parts. The first part essentially consisted of design services based on a fixed lump sum price which was signed on July 26, 2011, and executed on August 11, 2011. The design services consisted of schematics, 50% and final construction documents. This part also included geo-technical services. Ajax had a team of consultants including an architect, a structural engineer, etc., to design and produce the documents. The second part primarily consisted of the actual construction of the garage. For this part, Ajax had contracted with MAR Contracting, Inc. of Miami, FL to design, fabricate, deliver and erect individual precast structural elements to complete the structure. However, MDC, directly contracted with Coreslab Structures Miami, Inc. (Coreslab) of 10501 NW 121st Way, Medley, FL in Miami-Dade County, FL. Coreslab retained The Consulting Engineering Group, Inc. (CEG) of Chicago, IL to design the individual pieces and produce drawings for fabrication. MAR subcontracted with Solar Erectors to perform the erection of the concrete pieces. Coreslab contracted with Florida Lemark Corporation (Lemark) of Doral, FL to perform grouting, wash pour and other services. Coreslab had its own fabrication and concrete batching plant in Medley, FL.

MDC retained MEP Structural Engineering & Inspection Inc. (MEP) of Coconut Creek, FL to conduct threshold inspection of the garage during construction. MEP prepared inspection reports and distributed copies of the inspection reports to Ajax and MDC. MEP's scope of work

included inspection of structural element connections, welds, shims, washers, nuts, bearing pads and grout at the column splices and at the column bearings over the footings.

The Project

The project consisted of constructing a six-story garage with precast concrete structural members (columns, beams and double tees) with cast-in-place footings for the columns and precast shear walls, see Fig. 1 for plan and Fig. 2 for elevations. Double tees (two stems) spanned in the north-south direction supported by either inverted tee beams, known as IT beams, or by spandrel beams both running in the east west direction. In certain bays, double tees were supported over shear wall panels with ledges.

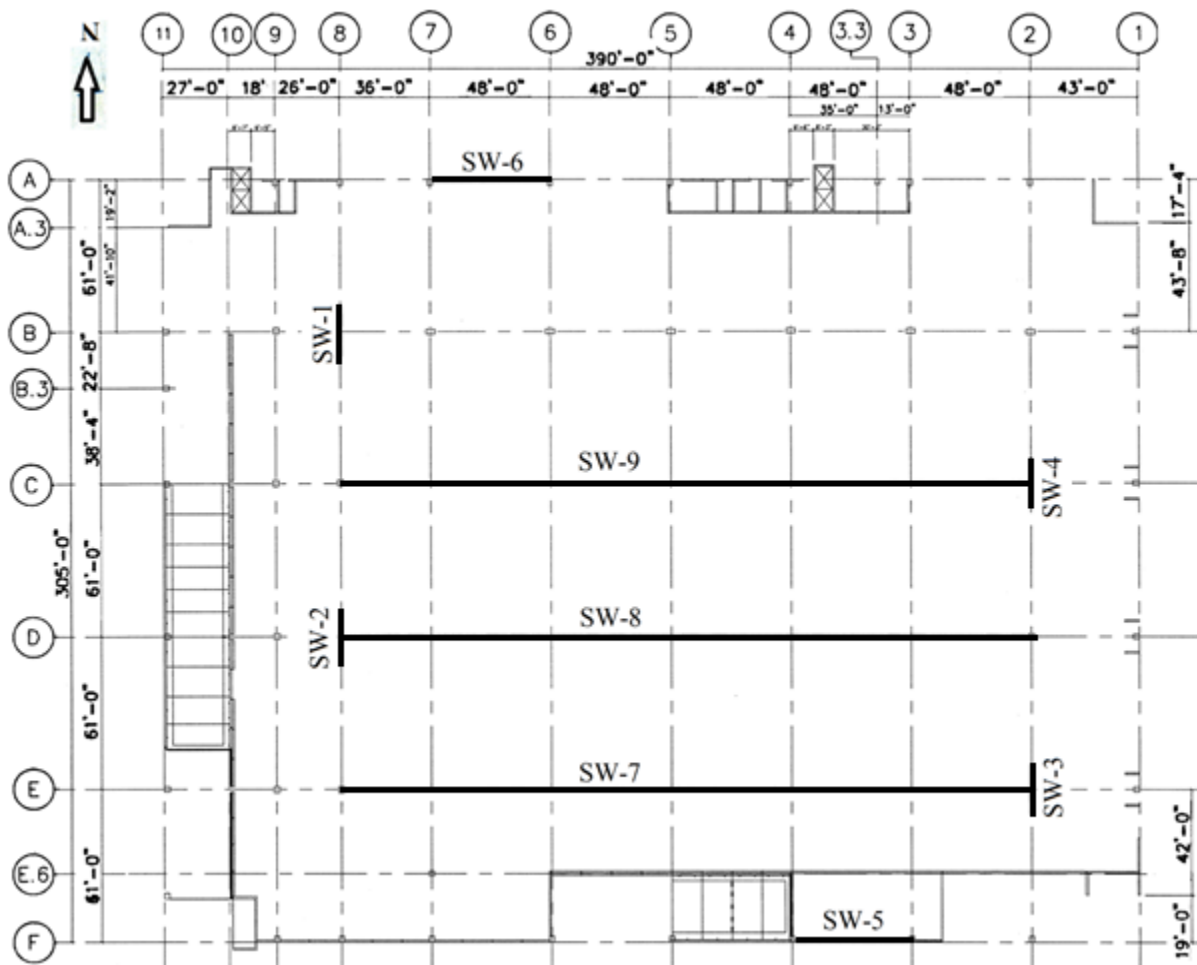


Fig. 1 – Plan of the garage 305' x 390'

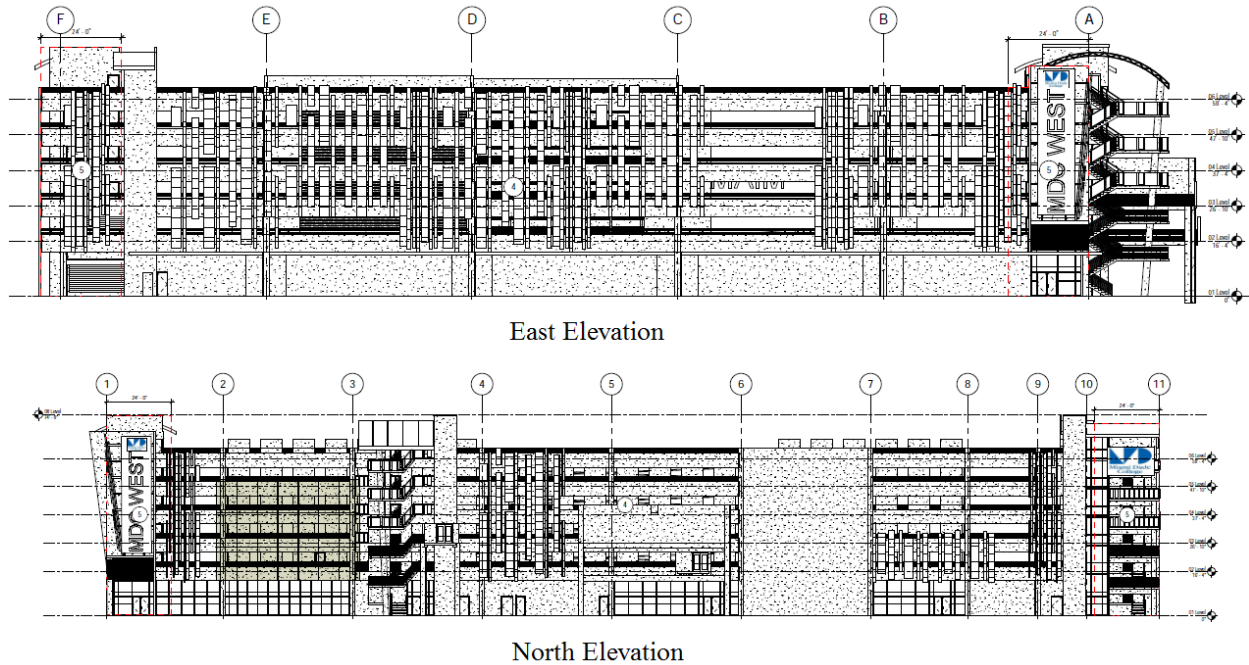


Fig. 2 – Elevations of the garage

The majority of the double tees were 12' wide, though some were less than 12'. The IT beams rested on corbels which were cast integrally with the precast columns. Generally, the double tees were 30" deep including 4" thick flanges. The ITs were provided with ledges where the double tee stems rested over Random Oriented Fiber (ROF) bearing pads. See Fig. 3 thru 6 for a typical detail of double tees, IT beams and spandrel beams. The double tees flanges were connected to each other at multiple locations to act as a diaphragm. The tees were also welded to the ITs and spandrel beams. The IT beams were welded to precast columns, see detail Fig. 5.

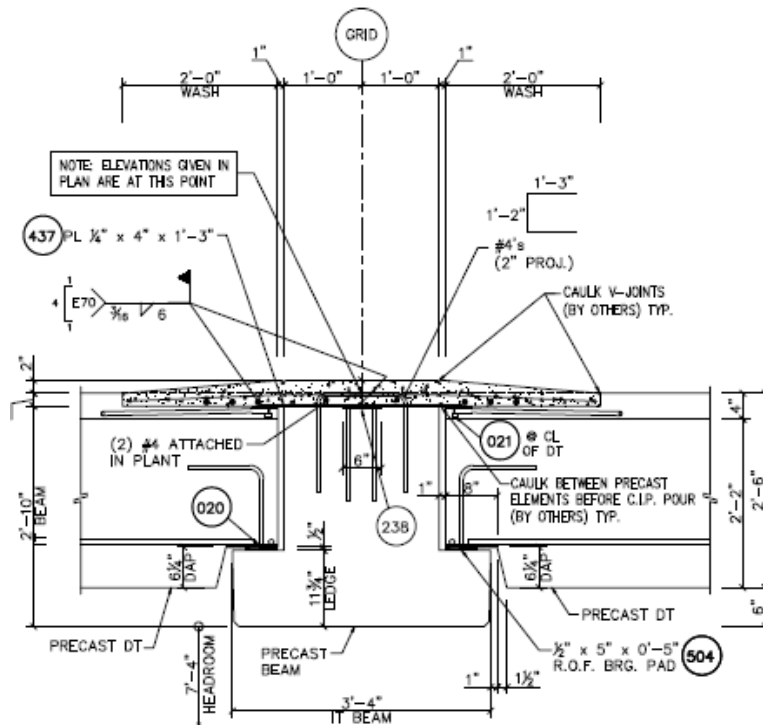


Fig. 3 – Typical connection detail of DT to IT beams

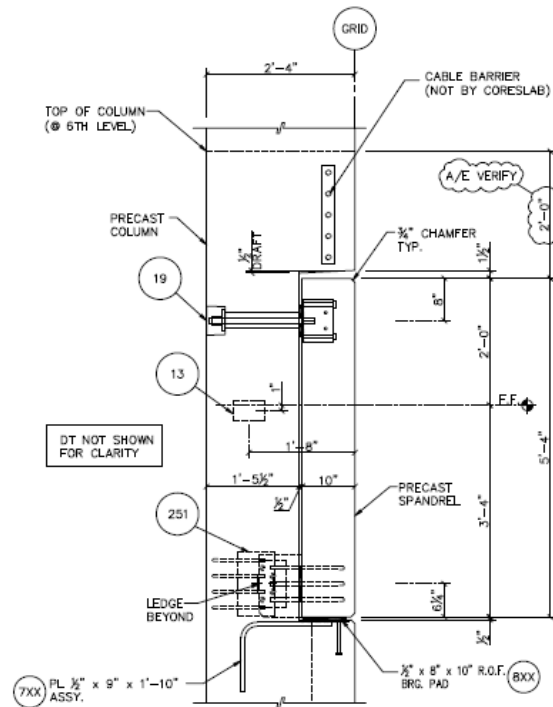


Fig. 4 – Typical connection of SP to column

DT - Double tee IT - Inverted tee beam SP – Spandrel beam

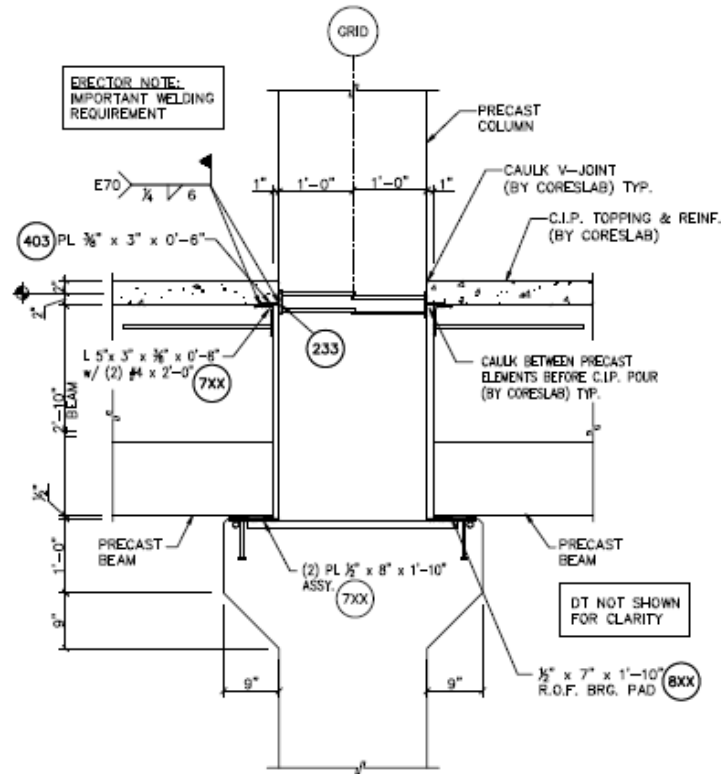


Fig. 5 – Typical connection of IT to column

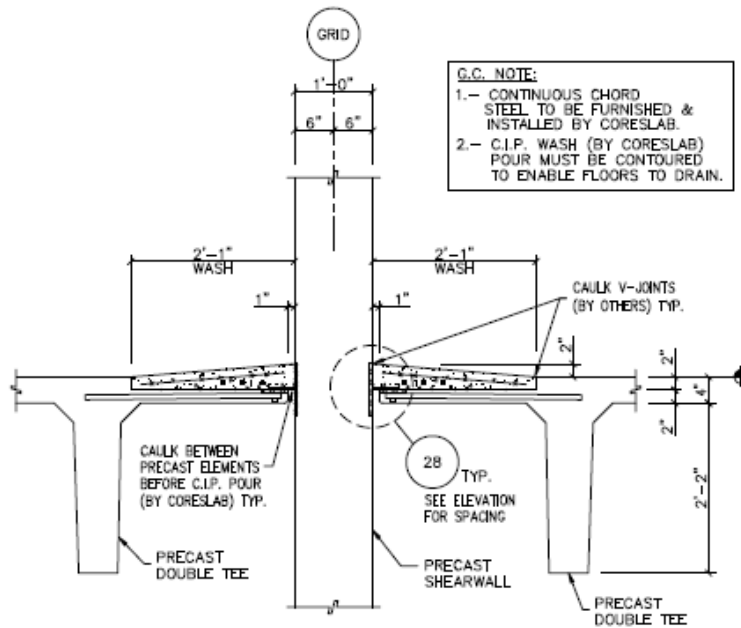
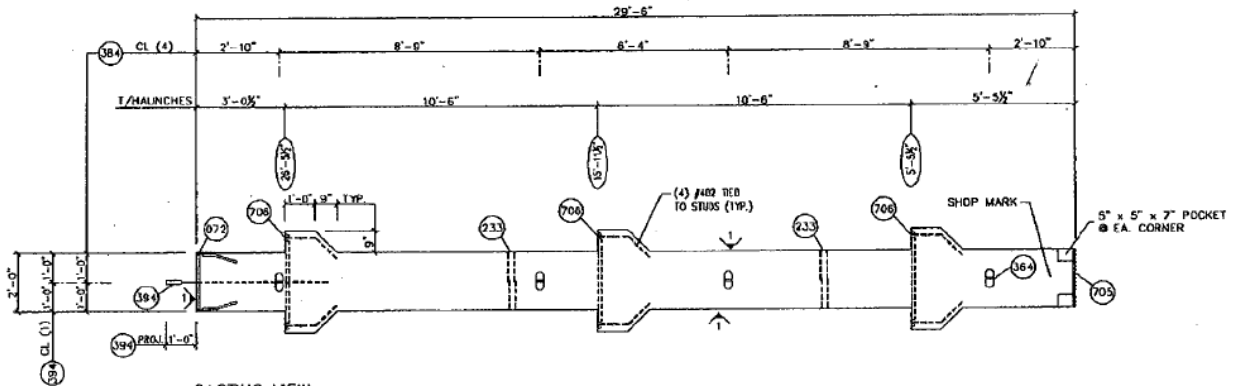


Fig. 6 – Typical connection of DT to shear wall

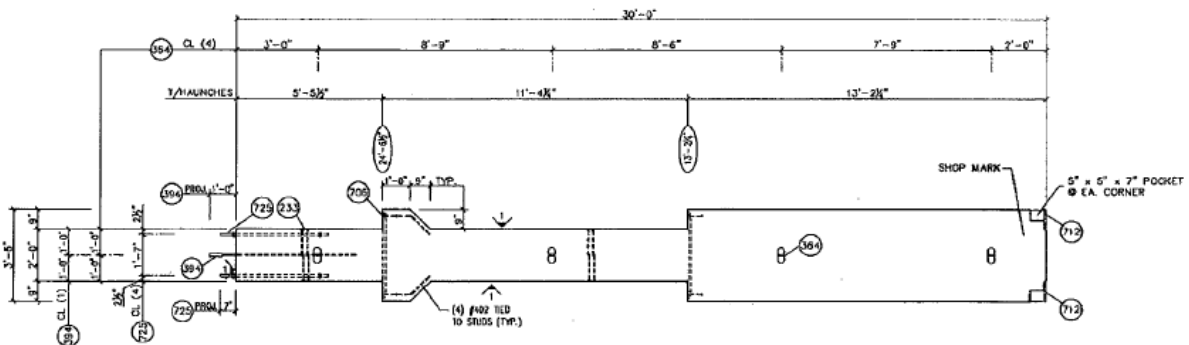
DT - Double tee

IT - Inverted tee beam

The precast columns were typically 24"x24" with corbels at each floor, see typical detail below, Fig. 7. The interior columns below the second floor were wider, 42" in the east-west direction but remained at 24" in the north-south direction.



B3 upper segment C112



B3 lower segment C111

Fig. 7 – Typical interior column

Except for the columns, all precast elements were prestressed. Columns were reinforced with only mild steel reinforcements, and were not prestressed. Columns were cast in two individual segments, the lower and upper pieces. First, the lower piece was erected, and loaded with floor beams and double tees. Then the upper piece was erected over the lower piece, and connected by four anchor bolts. The 2" space between the pieces were later grouted with high strength grout, see typical detail below, see Fig. 8.

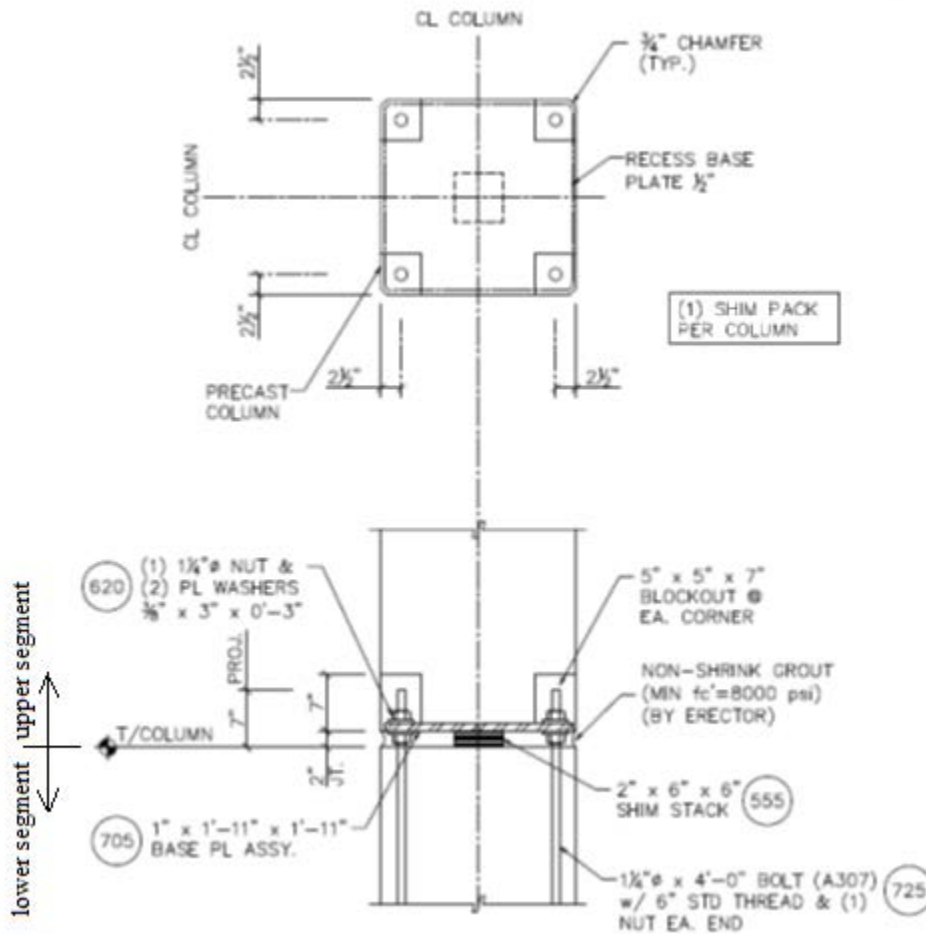


Fig. 8 – Typical column splice detail

Typically the concrete strength was 6,000 psi, although higher strength for some members was required by the design, discussed later. The typical bay of the garage was 48' in the east west direction, and 61' in the north-south direction. The floor-to-floor height was 10'-6" except 16'-4" for the first floor. The overall size of the garage was 305' x 390', and 62' high. The first floor was slab on grade poured over compacted soil. The typical cast-in-place footings for the interior columns was 23' x 23', and was 5 ft. deep, with 4,000 psi concrete. The precast columns were connected to the footings by four anchor bolts, and the space underneath the columns was later grouted with high-strength grout. See typical detail below (Fig. 9) for an interior column-footing joint. The anchor bolts were generally ASTM A-307 steel, and were 24" to 36" long.

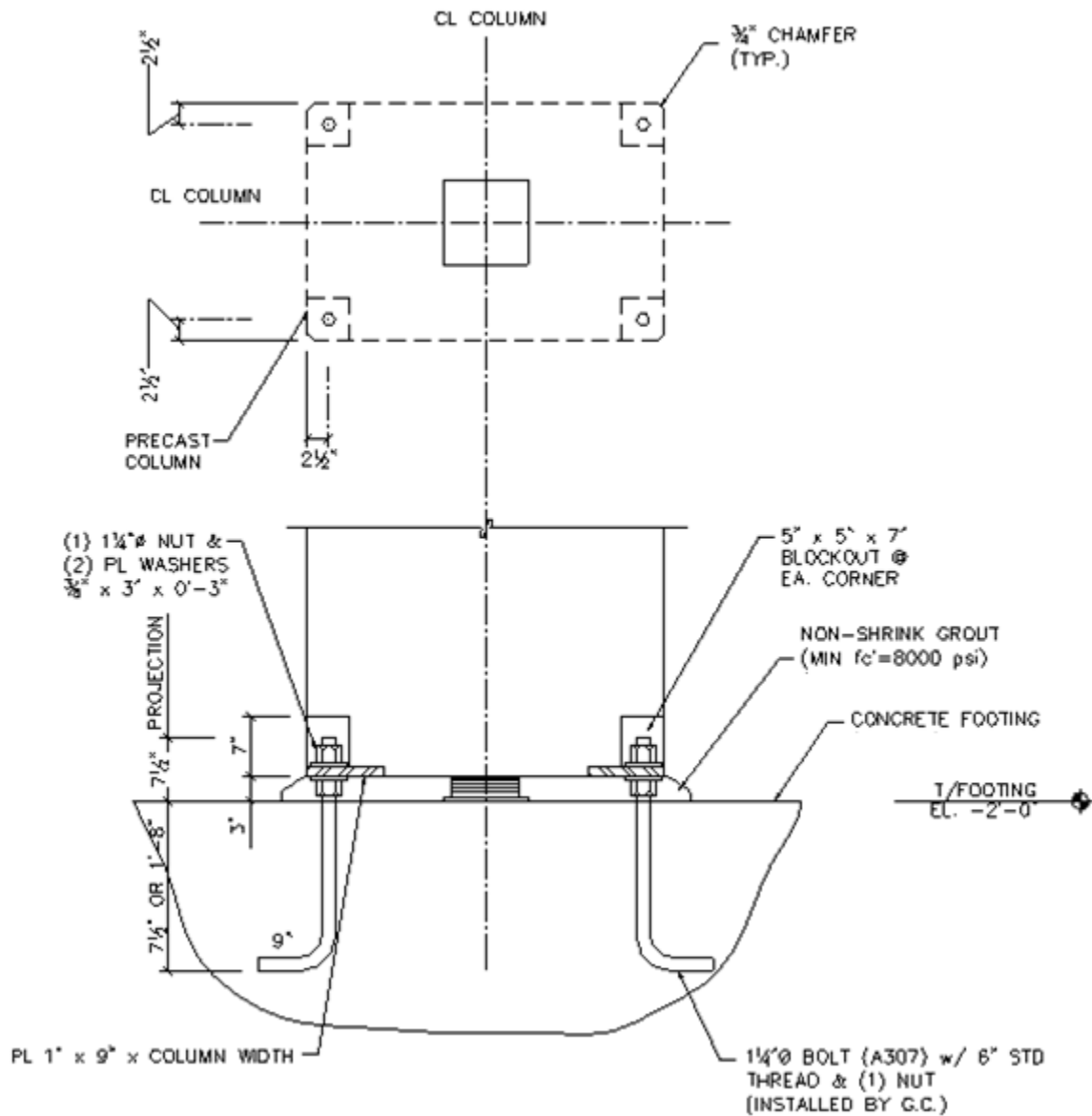


Fig. 9 – Typical detail at base of column

The lateral load resistance system was comprised of precast shear walls, known as wall panels, in north-south and east-west directions. Wind loads were considered for the lateral load. Seismic loads are not considered in Florida. The following were the key participants in the project.

1. The Board of Trustees of Miami-Dade College, Owner
2. Ajax Building Corporation, Design-Build General Contractor.
3. Harvard Jolly, Inc., Architects

4. Bliss & Nyitray, Inc., Structural Engineer of Record.
5. MEP Structural Engineering, Inc., Special Inspector
6. Coreslab Structures, Precast Manufacturer
7. Solar Erectors, Precast Erectors
8. Consulting Engineering Group (CEG), Specialty Engineer
9. Florida Lemark Corporation, Contractor for grout.

The Collapse

The collapse occurred at approximately 11:30 a.m. on Wednesday, October 10, 2012. At the time of the incident, the crane was loaded with a spandrel beam to be placed on the sixth floor between columns A2 and A3. The load was near, but not at, its final position when the incident occurred. The crane continued to hold the load even after the incident. The precast structural members erected on the same day and on days earlier suddenly collapsed to the ground from column grid line A to C in the north-south direction, and from column grid line 2 to 5 in the east west direction. The area impacted by the collapse measured approximately 132' in the east-west direction, and 122' in the north-south direction, see Fig. 10 thru 16 for aerial view of the collapse. All 111 double tees, from the sixth to the second floor between column grid lines A and C fell to the ground nearly directly below their original position, and pancaked over each other, see Fig. 19. One row of double tees immediately east of column grid line 5, between column grid lines A and C, did not fall but was badly displaced, skewed and leaning.

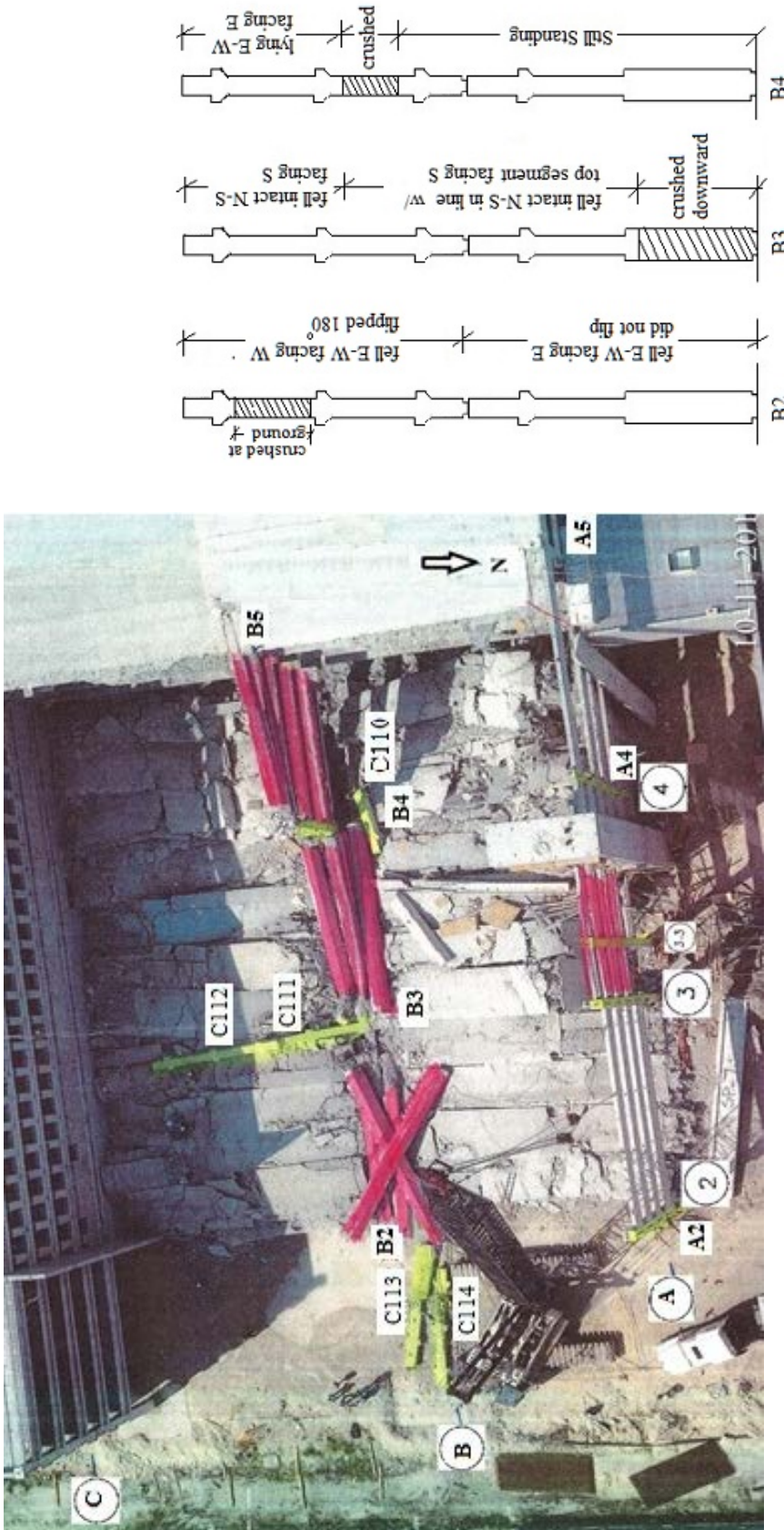


Fig. 10 – Aerial view of the collapse and details of columns B2, B3 and B4

Source for all aerial photos: Ajax

Index: C113 – lower B2 C111 – lower B3 C110 – upper B4
 C114 – upper B2 C112 – upper B3



Fig. 11 – Aerial view of the collapse looking west

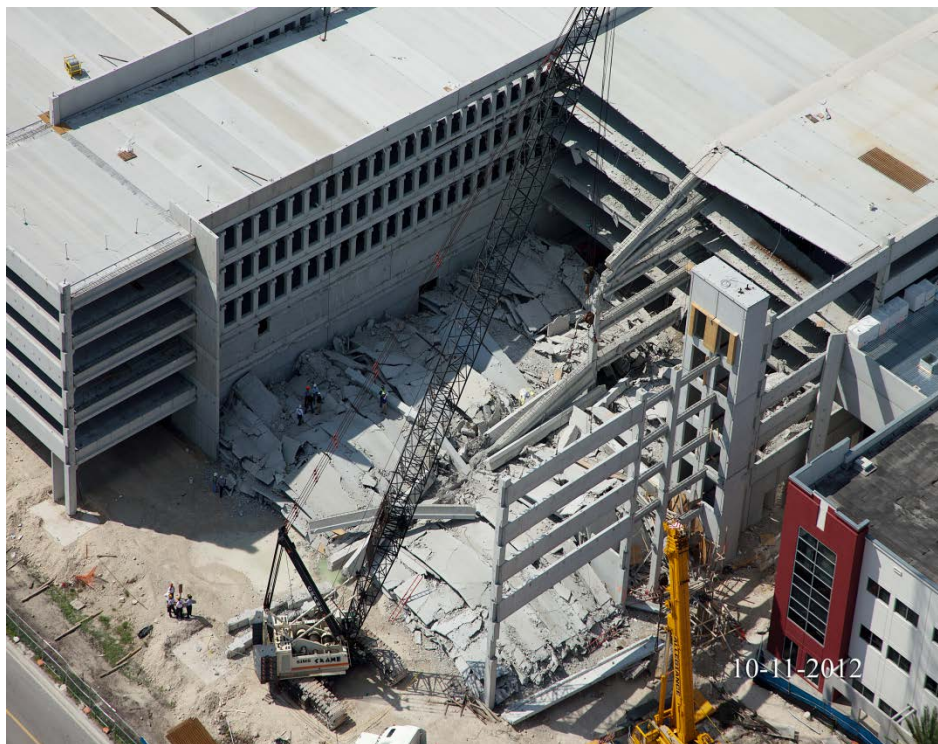


Fig. 12 – Aerial view of the collapse looking south

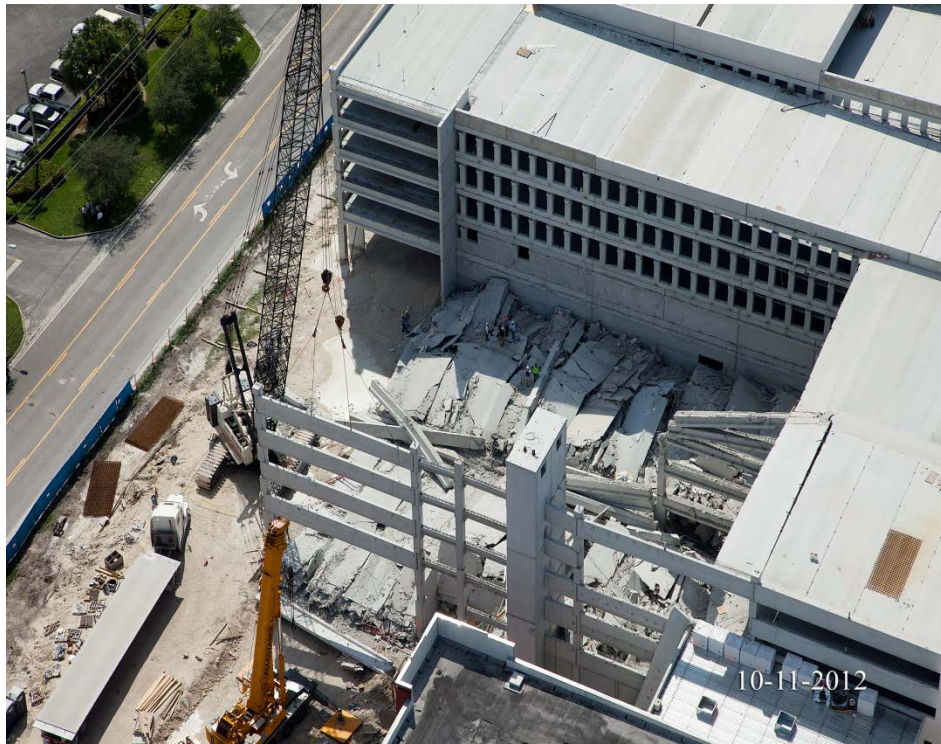


Fig.13 – Another aerial view of the collapse looking south



Fig. 14 – Another view of the collapse looking south



Fig. 15 – Aerial view of the collapse looking west



Fig. 16 – Another view of the collapse looking northwest

The collapsed double tees weighed approximately 3,000 tons. Two double tees north of grid line B and two double tees south of grid line B between columns B4 and B5 were also displaced. Ten IT beams between columns B2 to B4 from all floors, six to second, fell to the ground slightly skewed from their original position. The ten IT beams weighed approximately 250 tons. The sixth and fifth floor IT beams between columns B5 and B4 did not fall to the ground but sloped and leaned, and rested on the east end of the fourth floor IT beam. See Fig. 17.



Fig. 17 – Column B4 after the collapse

The upper and lower segments of columns B2 and B3 fell to the ground near their bases but skewed in different directions, see Fig. 18. The upper segment of column B4 above the fourth level fell to the ground, but the lower segment of column B4 and the remaining upper segment remained standing almost plumb, still attached to its footing. The splice between the upper and lower segments of column B4 remained intact. The weight of the B2 and B3 columns were approximately 45 tons. Thus, the total weight of concrete members collapsed into grids A to C and 2 to 4 was approximately 3,300 tons. The columns on the A grid lines remained standing but out of plumb, see Fig. 19.

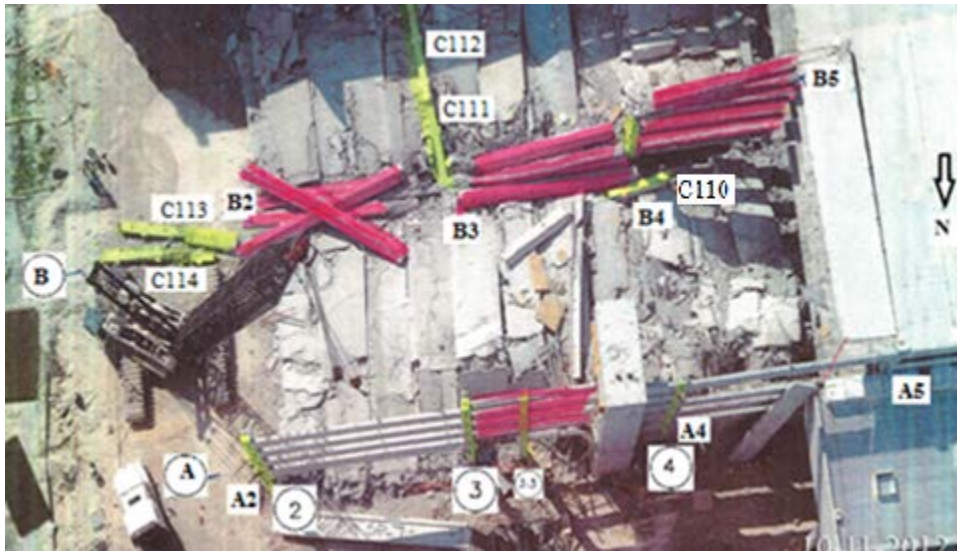


Fig. 18 – Orientation of columns B2 and B3 after collapse



Fig. 19 – Some of the fallen DT's

The fifth through third floor spandrel beams between column lines A-2 and A-3 remained in place, and did not fall. But the second floor spandrel beam fell to the ground towards the north. The sixth floor spandrel beam was still hoisted by the crane, and the crane did not release the

load. All four spandrel beams from the sixth to the third level between column lines A-3 to A3.3 remained intact except the second floor spandrel beam which fell to the ground. All five spandrel beams between column line A3.3 and the elevator shaft, and between elevator shaft and column line A-4 remained in place. All spandrel beams between column lines A-4 and A-5 remained in place except the fifth level spandrel beam which fell to the north.

The erection of the precast elements including double tees, beams and columns that fell took place at different times. Precast elements west of column grid line 3 were erected some three weeks before the incident, see Fig. 20. Erection of precast elements east of column grid line 3 took place in the last three to four days before the incident. Four hours before the incident, the ITs and some double tees on higher floors were placed between column grid lines 2 and 3. The extent of erection up to the time of the incident is shown in Figures 21 thru 28.



Fig. 20 – Pre-incident aerial photo (Progress as of 09/25/12)

Note B3 column was erected on 9/14/12, 26 days before the incident

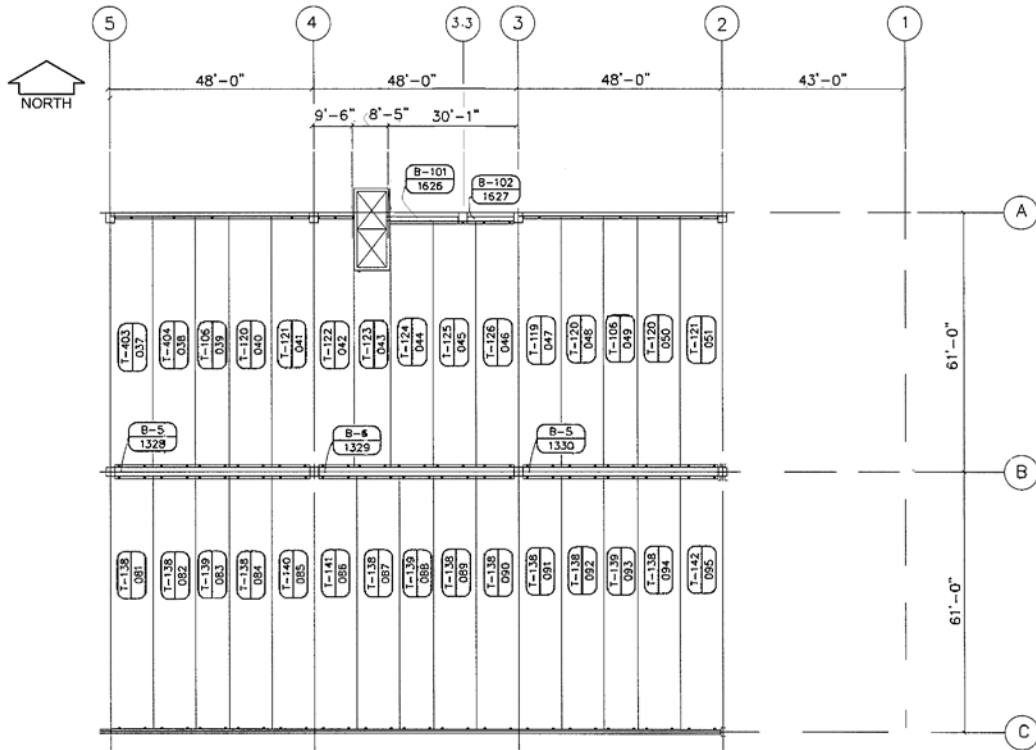


Fig. 21 – Level 2 Floor Plan (Progress as of 10/8/12 and 10/10/12)

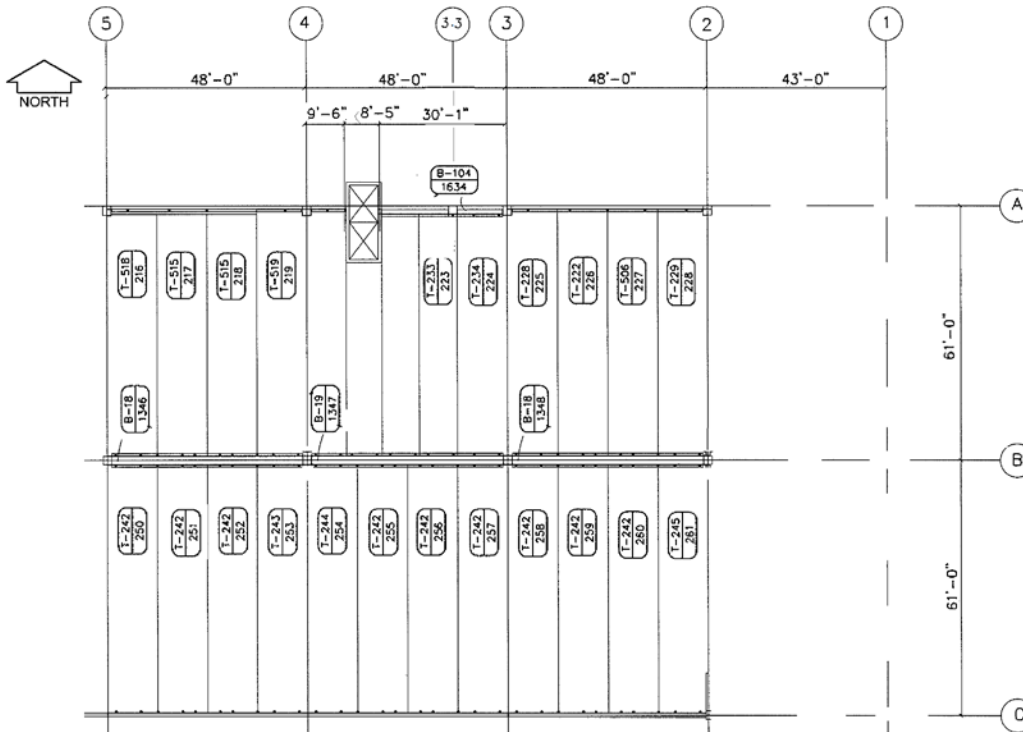


Fig. 22 – Level 3 Floor Plan (Progress as of 10/8/12 and 10/10/12)

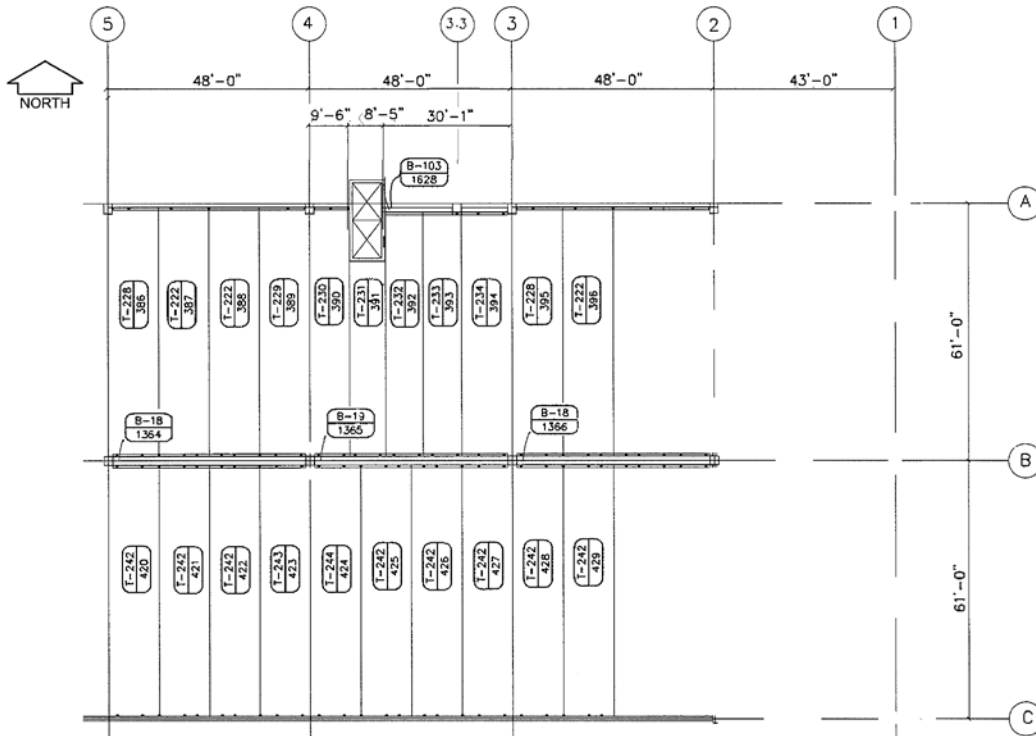


Fig. 23 – Level 4 Floor Plan (Progress as of 10/8/12)

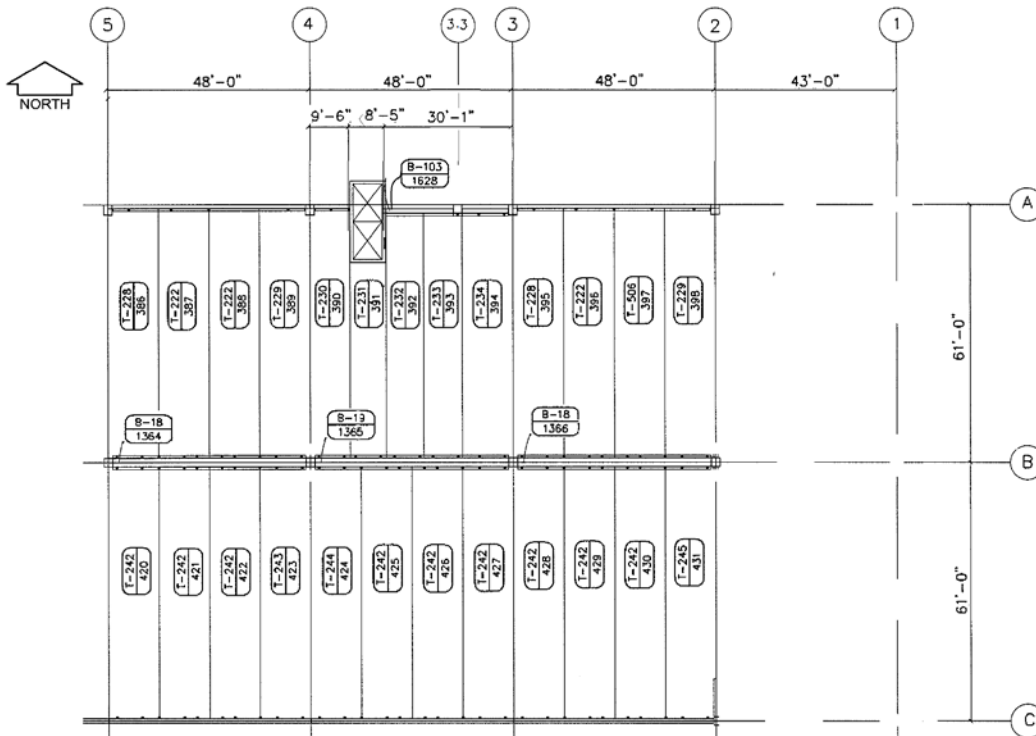


Fig. 24 – Level 4 Floor Plan (Progress as of 10/10/12)

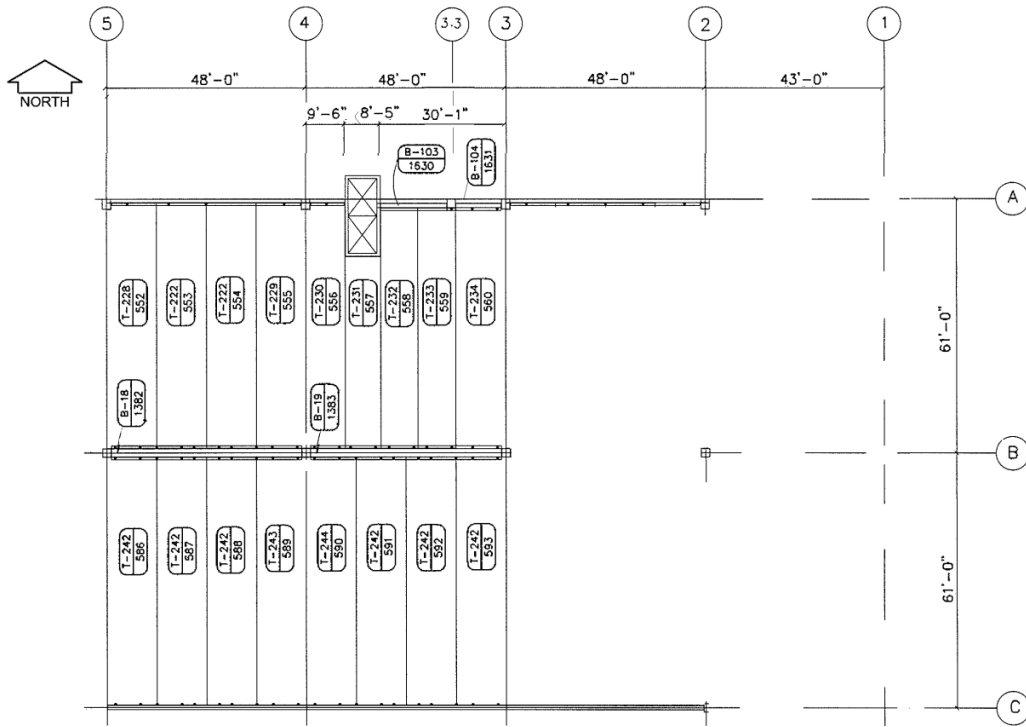


Fig. 25 – Level 5 Floor Plan (Progress as of 10/8/12)

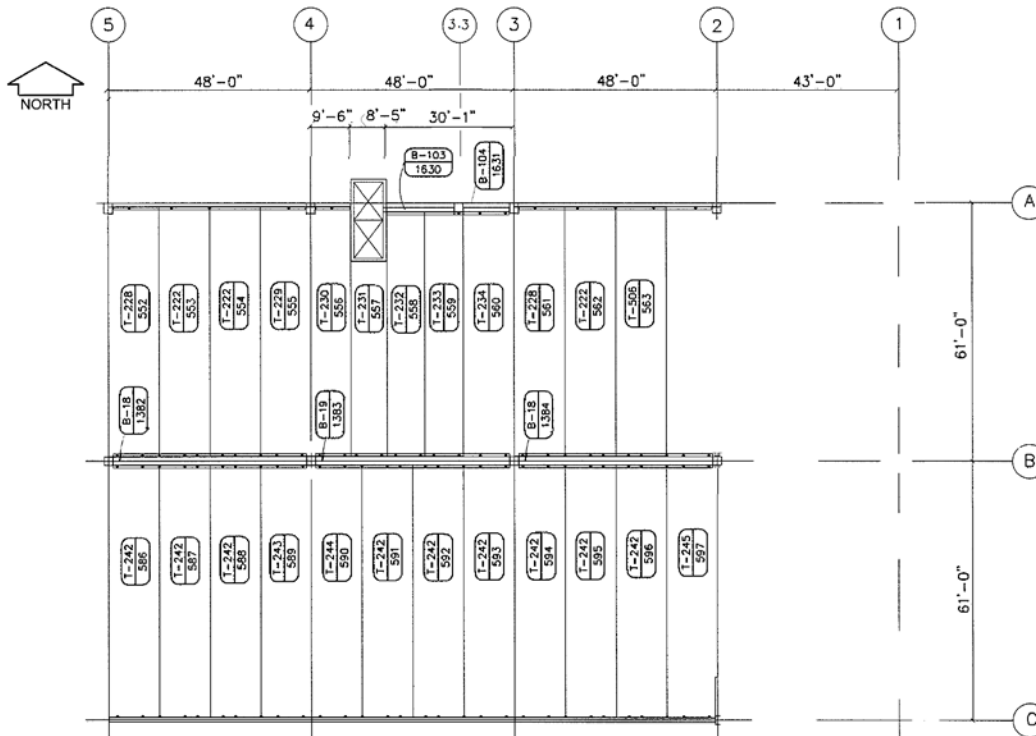


Fig. 26 – Level 5 Floor Plan (Progress as of 10/10/12)

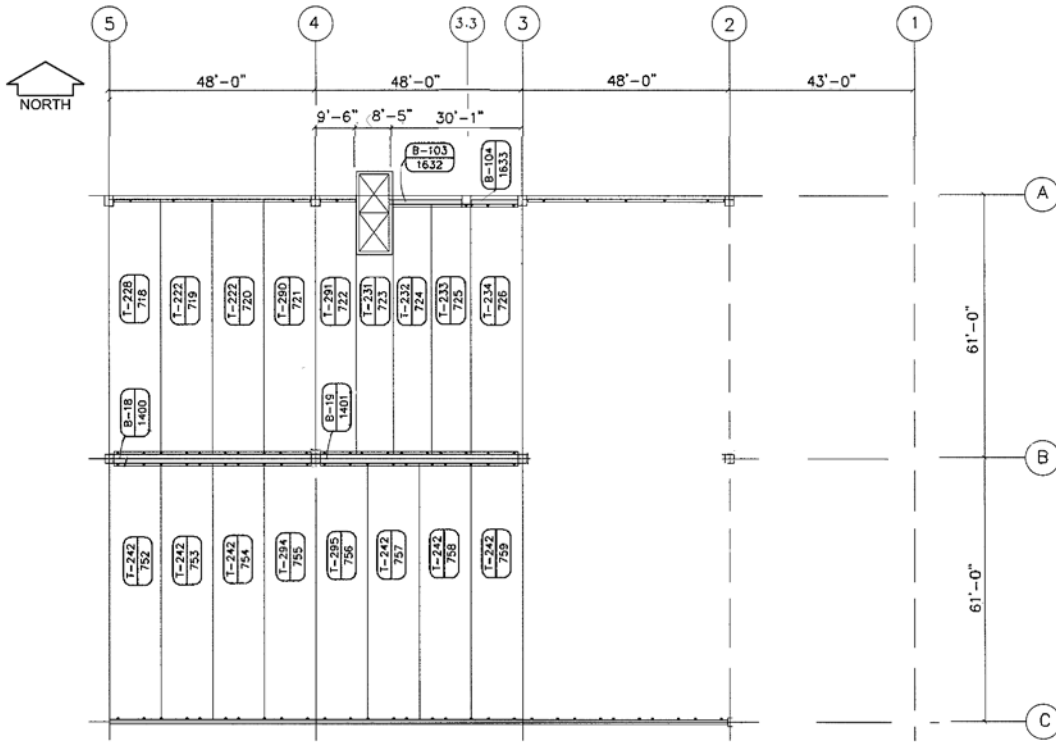


Fig. 27 – Level 6 Floor Plan (Progress as of 10/08/12)

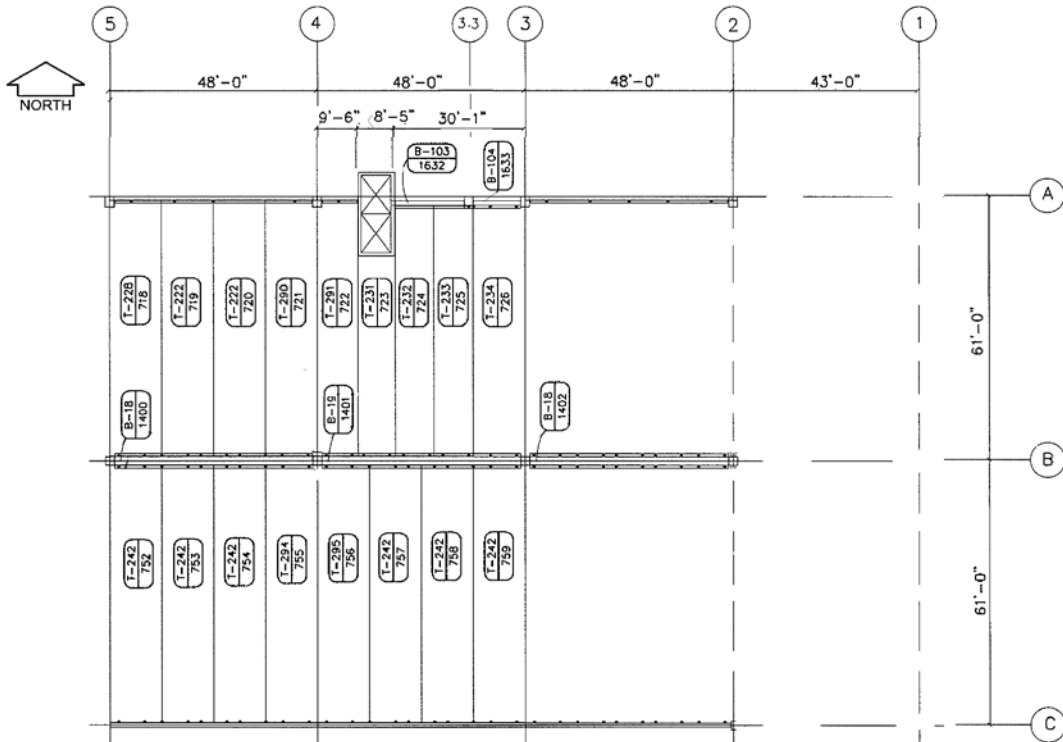


Fig. 28 – Level 6 Floor Plan (Progress as of 10/10/12)

Two employees of Solar Erectors, Inc., one employee of E&E Concrete Pumping Services Inc., and one employee of Stryker Electric Inc., were trapped under the massive falling concrete pieces and were killed. Three employees of Florida Lemark Corporation were injured.

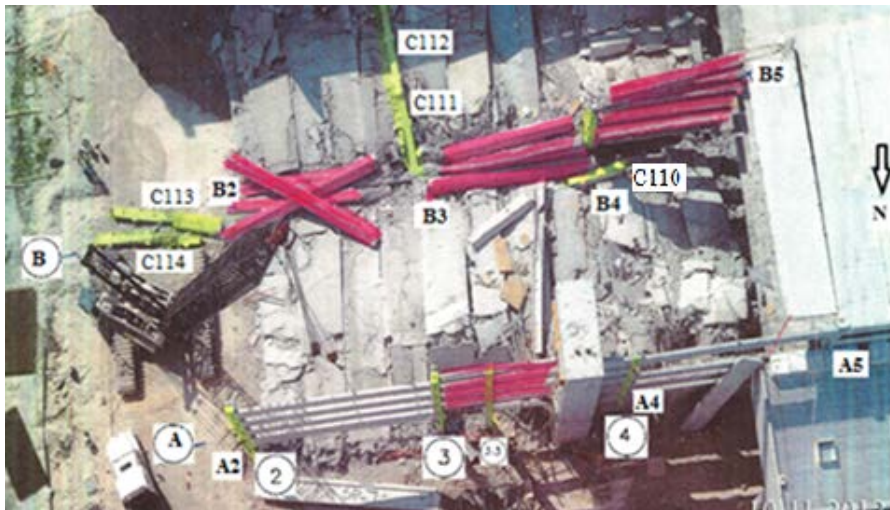
Detailed Field Observations

The detailed field observations were limited to the area of collapse, i.e., the area bounded by grid lines A to C, and grid lines 2 to 5. The precast columns were manufactured in two pieces, identified as upper and lower segments. The inverted T-beams are identified as BB23 6, specifying the location of the inverted T-beam at level 6 between columns B2 and B3.

Columns

Column B2 Upper Segment

The upper segment of column B2 flipped 180 degrees and fell to the ground in the east-west direction with the tip facing towards the base, approximately 15 ft. away, see Fig. 29. The upper and lower segments of column B2 separated at the splice below the fourth floor. The three corbels remained relatively intact. However, the column between corbel 6 and corbel 5 was shattered, exposing the longitudinal re-bars. This is believed to have occurred after the column hit the ground as broken pieces of concrete were visible near the shattered portion of the column. The upper segment was 24"x24".



C113 – lower segment

C114 – upper segment

Fig. 29 – Column B2 lying in the east-west direction

Two days before the incident, on October 8, 2012, the boom of a crane carrying a wall panel, and moving slowly from north to south (per Solar Erectors), struck the very top of the upper segment of column B2, see Fig. 30 and 31. Looking from the ground up, an engineer employed by Coreslab observed that the column splice was free of any cracks and distress, and therefore advised the erector to continue with the erection. The engineer did not perform any close examination in a man-basket or order any X-rays to determine whether any internal cracks resulted as a result of the bump. The next day the erector's transit indicated that the upper segment of the column was $\frac{3}{4}$ " out of plumb towards the south and west directions as a result of the bump, see Fig. 32. The erector's supervisor, however, recalled during an interview with OSHA that the column was reported by the foreman to be $1\frac{3}{4}$ " out of plumb. Regardless of the magnitude of the out of plumbness, the column was made plumb by wedges and shims when IT beams and double tees were placed on October 10, 2012. The Solar Erector's foreman said that the lower segment of column B2 was not plumbed after the incident with the crane. It is not known whether there was a need to plumb the lower segment.



Fig. 30 – Damage to boom when crane hit B2 column (pre-collapse)



Fig. 31 – Column B2 marks after crane hit



Source: Photo obtained from Miami Dade Police. Photo taken on 10/8/2012.

Fig. 32 – Column B2 out of plumb - after hit by crane

Field observation of the bottom of the upper segment of column B2 indicated that the reinforcement welded to the splice base plate was pulled off, and the embedded steel plate seceded and remained with the lower segment of the B2 column after the collapse.

Field observation indicated that the splice was grouted under the splice base plate. The three east corbels were free of any damage as there were no loads placed over them before the collapse. The three west corbels showed signs of distress when the IT beams fell off the corbel. The mid-section of the column between the sixth floor and the fifth floor corbel was shattered, believed to have happened during the collapse, see Fig. 33.



Fig. 33 – Column B2 shattered between level 6 and 5

Column B2 Lower Segment

The lower segment of the B2 column fell approximately in the east-west direction nearly parallel to the upper segment of column B2 except it did not flip 180 degrees. The bottom of the lower segment of the B2 column pointed towards the base. The two corbels remained relatively intact. The wider portion of the column remained mostly intact but the column above the second floor was badly damaged during the collapse, see Fig. 39. After the clean-up, see Fig. 34, it was discovered that there was 4" thick grout under the base plate of the column, although it did not cover 100% of the required area. Of the four anchor bolts projecting from the footing, the northeast and southeast anchor bolts were bent towards the east, while the other two anchor bolts remained nearly vertical, see Fig. 35. It was further noted that there were two stacks of steel shims, each containing six shims 6"x6".



Fig. 34 – Foundation of column B2 and B3 after clean-up



Fig. 35 – Base of column B2 after clean-up

The lower segment of column B2 was relatively intact except for the four anchor bolts at the top of the lower segment which was connected to the base plate embedded in the upper segment of

the column, see Fig. 38. The southeast anchor bolt slipped 4½" out of the column without substantially damaging the column, indicating complete bond failure between the anchor bolt and the concrete. There was a localized failure at the bottom of the anchor bolt where a nut was placed, see Fig. 36 and 37.



Fig. 36 – B2 lower segment C113



Fig. 37 – B2 lower segment C113



Fig. 38 – B2 lower segment C113

The southeast anchor bolt was bent toward the west. The bottom of the base plate was 6½" from the top of the concrete. The southwest and the northeast anchor bolts were also bent towards the west. The northwest bolt remained nearly vertical. The space below the base plate was grouted although it could not be ascertained whether it was 100% grouted over the entire area.

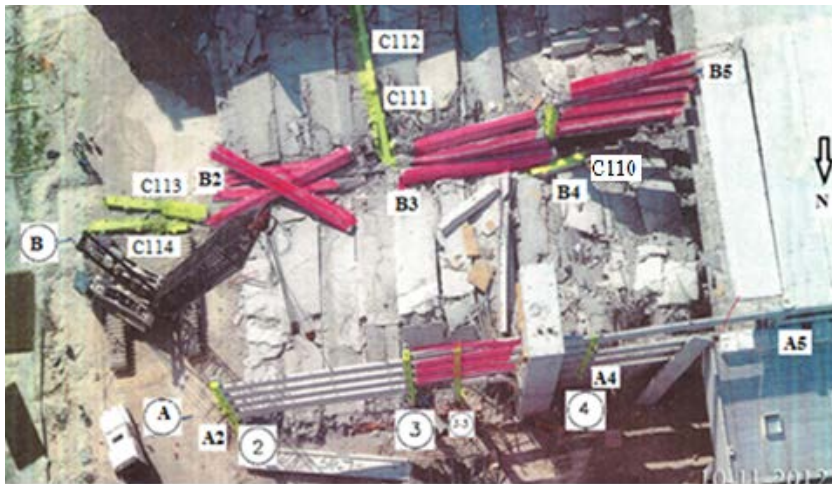
The column was 24"x24" except 42" (east-west direction) x 24" below the second floor bearings for the IT beams.



Fig. 39 – B2 lower segment C113

Column B3 Upper Segment

The upper segment of the B3 column remained intact, see Fig. 41 when it fell with little damage to the corbels or to the main body of the column. The column fell towards the south, see Fig. 40. As in the case of B2 column, the splice base plate embedded in the upper segment seceded and slid away with the lower segment of the B3 column after the collapse. The rebars welded to the splice base plate were sheared. There were clear indications that the sixth, fifth and fourth floor IT beams were welded to the column steel plates in both east and west directions. The column was 24"x24".



C111 – lower segment

C112 – upper segment

Fig. 40 – Column B3 lying north-south direction



Fig. 41 – Column B3 upper segment in the yard

Column B3 Lower Segment

The lower segment of the B3 column, which also fell towards the south, see Fig. 42, aligned with the upper segment, was unremarkable except for the fact that beginning four feet below the second floor IT beam bearing, the column was completely shattered and disintegrated into small pieces. All 8 longitudinal rebars were bent in a cage-like manner and completely exposed, see Fig. 43 and 44.



C111 – lower segment

Fig. 42 – Column B3 lower segment fell towards south



Fig. 43 – Column B3 lower segment



Fig. 44 – Column B3 lower segment

There were indications that the IT beams were welded to the column's steel plates at the second and third floors in the east and west directions. The four anchor bolts at the splice plates were intact but the splice base plate was bent. There were indications that there was grout at the splice between the upper and lower segments.



Note:
Splice plate remained
with lower segment C111

Fig. 45 – Column B3 lower segment splice

After the cleanup, see Fig. 46, it was discovered that grout was not placed at the footing underneath the column, but there was one stack of six shims 10"x10" placed under the column near its center, see Fig. 47. The shims left a faint mark over the footing without any deformation in the footing concrete. The four anchor bolts were bent like S curves, and were leaning towards each other in the north-south direction, see Fig. 48 thru 51. The two 9"x24" base plates were bent, see Fig. 47. The footing remained leveled within an inch at the four corners.



Fig. 46 – Base of B3 cleanup



Fig. 47 – B3 column base plate and shims



Fig. 48 – Bent anchor bolts

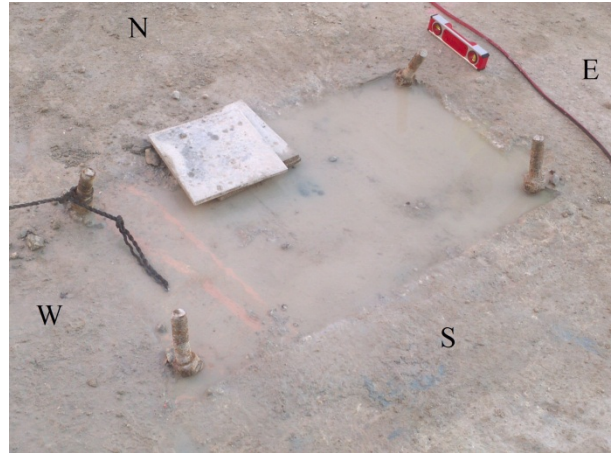


Fig. 49 – Anchor bolts bent towards N-S



Fig. 50 – Deformed SW anchor bolt

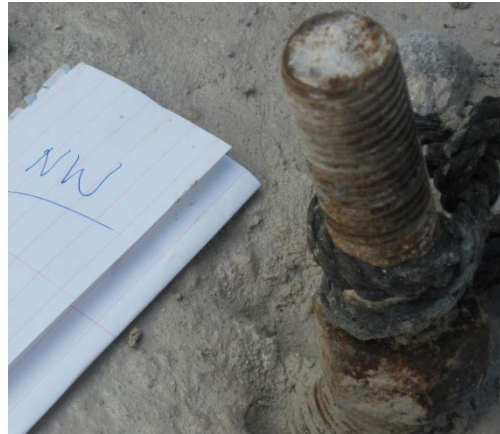


Fig. 51 – NW anchor bolt

Column B4 Upper Segment

The upper segment of column B4 disintegrated between the fifth level and fourth level corbels. The fourth floor splice remained intact. The remaining portion of the column fell to the ground towards the east-west direction, see Fig. 40. When the column partially disintegrated above the fourth floor and below the fifth floor corbels, see Fig. 53 thru 55, the IT beams between columns B-4 and B-5 supported over the sixth and fifth floor corbels fell over the fourth floor IT beam, and remained there after the collapse, see Fig. 52. The remaining portion of the upper segment remained connected to the lower segment through the splice. The column was 24"x24" reinforced with four #11 bars.



Fig. 52 – Column B4 remained standing after collapse



Fig. 53 – Column B4 upper segment above splice after demolition



Fig. 54 – Column B4 upper segment



Note: Section missing below the fifth floor corbel

Fig. 55 – Column B4 upper segment

Column B4 Lower Segment

The lower segment of column B4 did not fall and remained standing well after the collapse, see Fig. 52. Attached with the lower segment through the splice was the partial upper segment of the column from the location of the splice to the midsection between the fourth and fifth corbels.

The column was nearly plumb within an inch. The splice between the segments was grouted for $\frac{3}{4}$ of the area but $\frac{1}{4}$ of the area did not appear to have been grouted, see Fig. 56 and 57. Also, smaller shim plates were used at the splice. The four anchor bolts at the splice remained intact, and had to be burnt during the removal phase. The third floor corbel was unremarkable. The column became wider below the second floor to 42" (E-W) x 24" (N-S). The lower segment was undamaged.

There were weld remnants to indicate that the IT beams were welded to the steel plates embedded in the column at all floors. After the cleanup it was confirmed that there was grout underneath the column at the footing level. The grout measured approximately 4" high, see Fig. 58 and 59.



Fig. 56 – Grout at splice of B4 column



Fig. 57 – Close-up of the grout at splice of B4 column



Fig. 58 – Base of column B4



Fig. 59 – Grout at base of Column B4

Column A2

The A2 column remained standing, but was out of plumb. The second floor spandrel beam became disconnected and fell to the ground nearly parallel to the east-west direction. Apart from being out of plumb, there was no remarkable damage to the column. After the cleanup, it was discovered that the base of the column was fully grouted at the footing level. Also two sets of shim plates were used. The two north anchor bolts were intact and not bent. However, the southeast and southwest anchor bolts were bent and leaning towards the south. The anchor bolts were most likely bent during the removal phase.



Note the grout at the base

Fig. 60 – Footing for column A2

Column A3 and A3.3

Columns A3 and A3.3 had a combined footing due to proximity. A3 and A3.3 also remained standing, and were out of plumb. The second floor spandrel beam lost its support and fell to the ground. The two columns were mostly undamaged, and had a common footing. After the cleanup it was discovered that the underside of column A3 was grouted at the footings, see Fig. 61. Also, shim plates were used at the center of the column. All four anchor bolts of column A3 were leaning towards the north. For the A3.3 column, there was no grout under the column, see Fig. 62. The four anchor bolts of the A3.3 column were bent and were leaning severely towards

the south, almost 30 degrees to the horizontal. It is believed that the anchor bolts were bent during the demolition/removal process.

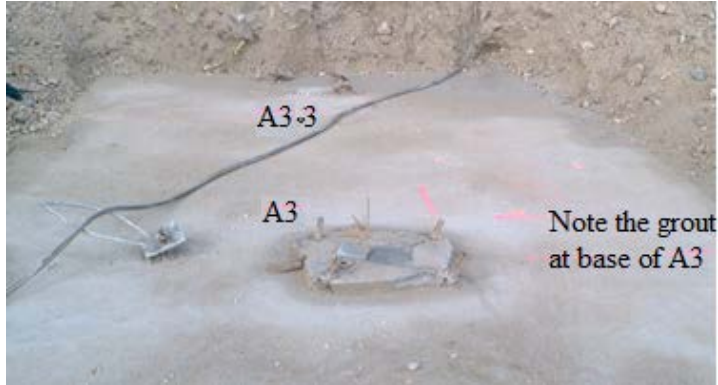


Fig. 61 – Columns A3 and A3.3



Fig. 62 – Base of column A3.3. No grout

Column A4

Column A4 remained standing after the collapse but was out of plumb, 11½" towards the north, and 5½" towards the west. Besides, A4 column was cracked near the splice in the upper segment. The fifth floor spandrel beam fell during the incident and ended up standing upright leaning against the other spandrel beams, see Fig. 63.



Fig. 63 – Column A4

IT Beams

Generally speaking, close examination of the IT beams, see Fig. 64 thru 71, indicated that the IT beams were welded at both ends to the columns, at least partially, approximately three inches as a minimum at all locations. With regard to the welds of the double tees to the IT beams, the field observations indicated that there were certain beams to which the double tees were not welded. Most notable were the fifth and fourth floor IT beams between columns B2 and B3. The contract documents required that welding of connections shall follow closely behind the erection of the units, see appendix A-14. At places, they were welded at two locations out of four, e.g., IT beam on the third floor between columns B2 and B3; and IT beam on the second level between columns B3 and B4. The IT beam on the third floor between columns B3 and B4 was welded on one side only. The IT beam on the second floor between columns B2 and B3 was welded at fewer locations. Similarly, the IT beam on the sixth floor between columns B3 and B4 was welded on one side only.



Fig. 64 – IT beam between B2 and B3
3rd level in the yard



Fig. 65 – IT beam between B2 and B3
4th level in the yard



Fig. 66 – IT beam between B2 and B3
5th level in the yard



Fig. 67 – IT beam between B2 and B3
6th level in the yard



Fig. 68 – IT beam between B3 and B4
2nd level in the yard



Fig. 69 – IT beam between B3 and B4
3rd level in the yard



Fig. 70 – IT beam between B3 and B4
4th level in the yard



Fig. 71 – IT beam between B3 and B4
6th level in the yard

Footings

The foundation of the columns consisted of cast-in-place individual footings except for the A3 and A3.3 columns which had a combined footing. The footings were generally 23'-6" x 23'-6" x 5'-0" deep, see Fig. 72. All footings were determined to be within tolerance levels regarding their size and levelness. The footings generally were cast with 3,000 psi concrete. No undue settlements were detected.



Fig. 72 – Exposed foundation for column B3

Production of Precast Elements

All precast elements were produced by the Coreslab Structures Miami, Inc. manufacturing plant located in Medley, FL in Dade County. The plant is located a few miles from the construction site. Coreslab Structures Miami, Inc. is a subsidiary of Coreslab International, Inc. which also owns several subsidiaries manufacturing precast elements in several states in the U.S. Solar Erectors US, Inc. also falls under the umbrella of Coreslab International, Inc.

As discussed earlier, Coreslab retained CEG of Chicago as a Specialty Engineer to furnish actual design of individual members, e.g., columns, IT beams, double tees, shear walls, spandrel beams,

wall panels, etc. CEG's design was based upon the criteria and information provided by the structural engineer of record. CEG has been designing precast elements for Coreslab for over fifteen years. Detailed drawings and computations were sent to the structural engineer of record for approval, after which they were scheduled for fabrication in the order required by the erector. Double tees and IT beams were manufactured in a "line" assembly, seven at a time, and they were prestressed. Columns were manufactured one at a time and were not prestressed.

Coreslab has its own concrete batching plant to produce concrete using one of the three design mixes approved by the structural engineer of record. Testing of concrete was conducted as per ACI requirements. Coreslab used the design summary sheet prepared by CEG to manufacture the precast elements. For some of the precast elements, CEG inadvertently indicated concrete strengths lower than what was required by the design. This was discovered by Coreslab after the incident, and is discussed in detail later in this report.

An engineer from Coreslab frequently visited the construction site to coordinate the erection and delivery schedule, and to resolve issues in consultation with CEG, at times on a daily basis. The Coreslab engineer was informed by the erector of the strike on the top of the B2 column by the crane boom. After having examined the fourth floor splice from the ground the next day, the engineer asked the erector to proceed with the erection as discussed earlier. The engineer saw no visible crack at or near the splice from the ground.

Erection of Precast Elements

Solar Erectors US Inc., a subsidiary of Coreslab International, Inc. was the designated erector. The erection of the precast elements started approximately in July 2012. There were six employees in the "raising gang". Solar rented two Sims cranes initially but this was reduced to one by the time of the collapse. There was an erection foreman at the site everyday who guided the raising gang. The erection began at grid line 11 and A at the northwest corner. Typically, two bays would be erected with all precast elements, e.g., IT beams, double tees and spandrel beams placed up to the roof level. After the two bays were completed, then the next two bays would be begun. There were two transits, one for the north-south direction, and the other for the

east-west direction. Initially, the columns would be plumbed by means of the four leveling nuts before any horizontal members were placed. If there would be a need to re-plumb the columns later, which often occurred during erection, wedges and shims would be placed between the IT beams and columns, and the IT beams and double tees. Leveling nuts would not be re-adjusted.

There were two crews of welders who would follow the erectors to weld the IT beams to the column, the double tees to the IT beams, and the flanges of the double tees. The first crew would weld approximately 3", and the rest of the weld would be completed by the second crew which followed behind the first crew. The crane would not release the load until the weld by the first crew was completed, although it was discovered later that this was not true in all cases. For example, post-incident observation indicated that the double tees were not welded to the IT beams on the fourth and fifth levels between column B2 and B3, discussed earlier. At times, there was a time lag of two days between the crews. The normal practice was to weld the double tee to the IT beam at each end of the double tee, and at least two or four connections between the flanges.

Although the Solar Erector foreman stated that it took approximately 30 minutes or longer to erect either an IT beam or a double tee, the erector on the day of the incident erected two IT beams on the fifth and sixth floor between columns B2 and B3, and at least eleven double tees which would normally have taken 6½ hours. This was accomplished in less than four hours, showing an urgency to complete the work as fast as possible.

When asked why the B2 column was not braced on the day of the incident, the Solar Erector foreman said that the B2 column was already braced by the IT beam on all floors in the east-west direction, and by double tees on all floors except on the sixth floor in the north-south direction.

The foreman recalled that he was only a few feet from the A2 column, and was confident that the collapse initiated from the west and proceeded towards the east. The foreman said that the east bay was the last to fall.

The Solar Erector's foreman had a supervisor who would occasionally visit the site, and he essentially depended upon his foreman to complete the job. The supervisor had little input on the erection at this site.

Threshold Inspection

Under Florida statute 553.71, the garage structure was classified under threshold Structures, and therefore required a special inspection by a certified special building inspector according to the Florida Building Code. The Miami Dade College authorities accepted the proposal from MEP Structural Engineering, Inc. (MEP) to perform the services of code review, threshold inspection and code inspections. The scope of their work under threshold inspection was detailed in the construction document S1.02 prepared by the architect and the structural engineer of record. The principal threshold inspector was required to be a licensed professional engineer, but his authorized representatives who performed under the responsible charge of the threshold inspector were not required to be an engineer. MEP's threshold inspectors were Mr. Otto Letzelter, PE, and Mr. Hector Vergara, PE. The authorized representative at the site was Mr. Eduardo Martin. Martin was at the site practically every day from 8:00 AM until the close of business. For the final inspection of welds and items that needed engineering evaluation, Martin would call either Letzelter or Vergara for assistance and guidance. For all other items, Martin would conduct the daily inspections himself and write a daily report commenting on the items he had inspected either the same day or the day before. Martin's reports would generally be stamped and signed by Vergara. The reports were sent to the owner, general contractor and others.

Martin or his supervisors did not inspect temporary bracings, only the permanent elements which included all connections between beams and columns; double tees to beams, spandrels and wall panels; flange connections between double tee flanges; splice connections of the columns; connections of the columns to the footings; shims, grouts, anchor bolts, etc. Interestingly, all inspections of the column splices were conducted from the ground outside of the safety lines, sometimes from as far as 40 feet away. With the splices located some 30 feet above the ground, visual inspection could not be considered reliable. Martin did not request a man basket to conduct inspections. Inspection reports of the upper segment of columns B2 (C114), B3 (C112)

and lower segment of column B2 (C113) are attached, see table below and appendix A-10 and A-11. The inspection reports on the column B3 lower segment (C111), and column A3.3 lower segment (C135) where the grout between the footing and the column was discovered to be missing after the collapse, are not available. In fact, there are eighteen columns where there are no reports of inspection of the grout. They are: A2, A3, A3.3, A11, A.3-11, B3, B6, B10, B11, C9, C11, D10, D11, E10, E11, F5, F7 and F8. It is not certain whether there are grouts at all of these locations, but it is certain that there were no grouts at B3 and A3.3.

| Column | ID | MEP Inspection date | MEP Report # |
|------------------|------|---------------------|---------------|
| B3 lower segment | C111 | not available | not available |
| B3 upper segment | C112 | 8/7/2012 | 1082 |
| B2 lower segment | C113 | 10/8/2012 | 1233 |
| B2 upper segment | C114 | 10/8/2012 | 1233 |

Note: see MEP Structural Inspection report in appendix A-10 and A-11.

As a representative of the Special Inspector, Martin did not appear to be well versed with the process of how erection proceeded at the site on a number of issues. When asked whether the contractor would place the grout under the column base plate at the footing level before or after the IT beams were erected, Martin expressed a lack of knowledge. When asked whether the contractor would place welds connecting the IT beams to the columns immediately after erection of the IT beams, Martin expressed ignorance. When asked whether he would examine the transit readings of the columns to ensure plumbness, he said that he never saw them. When asked whether the contractor would complete welds of the double tees to the IT beams at the lower floors before completing the erection of precast elements at the next higher floors, Martin expressed his inability to answer the question. Martin was on the site on October 8, 9 and 10 of 2012. Martin did not know that the boom of a crane had hit the top of the column B2 on October 8, 2012. In fact he reported that he heard of the incident after the collapse. He also stated that

some of the information that he entered in his reports was based upon what others told him, which he would verify later. Martin said that he did not issue any letter of non-compliance on any matter on this project.

Letzetler, PE of MEP visited the site approximately 10 times during the course of construction. The primary reason of his visits was to inspect permanent welds, as required by Miami-Dade County that a professional engineer examine all permanent welds. He noticed deficiencies in approximately 10% of the welds which were later corrected at various times. A majority of the deficient welds needed additional pass. A few had to be redone. Letzetler stated that the welds were substantially complete from the 2nd to the 5th floor up to 10 feet east of column line 5. Letzetler stated that MEP was required to check the plumbness of the column at the completion of the project.

The Consulting Engineering Group, Inc.

Coreslab retained The Consulting Engineering Group, Inc. (CEG) of Prospect, IL, as a specialty engineer, to design the individual precast pieces, draw details of the connections and prepare erection drawings of the various precast elements. These details and drawings were meant to be used by Coreslab to fabricate the precast elements, and to be used by the precast erectors for placement of the individual pieces. These drawings did not indicate the temporary bracings to maintain stability during construction until all the elements were in place.

CEG was responsible for the adequacy and conformance of the design to the applicable building codes. Although the design was forwarded to the structural engineer of record for his approval, the responsibility for the design rested with CEG. There were no major comments by the structural engineer of record on the submissions by CEG. The structural engineer of record is responsible for the overall design after the building is completed. To facilitate fabrication and erection, CEG marked every precast element with two numbers. The first number was the piece mark, and the second the control number, e.g., C-111 (1269). There could be more than one element with the same C-111 mark in the project, but the control number was a unique number assigned to a specific location where the elements would be actually erected.

During the fabrication of precast elements, CEG and Coreslab frequently consulted each other to clarify things and make some minor changes to suit the fabrication process. There was one major issue, however, that we discovered during the investigation. It appears that this issue was already known to CEG and Coreslab, but only after the collapse. For columns B3 through B-6, the required concrete design strength was higher than the strength of the concrete employed in fabricating them. This occurred because of an error on the part of CEG personnel who mistakenly indicated 6,000 psi as the design strength of the concrete. CEG reanalyzed the columns and came to the conclusion that 6,000 psi for the upper piece of the column would be adequate. However, for the lower piece of the columns, 6,000 psi would not be adequate. A minimum of 7,100 psi would be required. The 28 days break of the concrete specimen showed that the concrete had an actual average strength of 7,200 psi (see table below and appendix A-4 to A-9), which is higher than the required design strength of 7,100 psi. Therefore, CEG concluded after the collapse that the intent of the design was met. Moreover because the column was erected some 56 days after it was fabricated, the concrete was expected to gain at least 10% higher strength than 7,200 psi

| Column | ID | Date Poured | 1st Actual Reading (psi) | 7 days strength (psi) | 28 days strength (psi) lowest value |
|------------------|------|-------------|--------------------------|-----------------------|-------------------------------------|
| B3 lower segment | C111 | 6/26/2012 | 3382 (12 1/2 hrs) | 6246 | 7081 |
| B3 upper segment | C112 | 5/29/2012 | 3516 (14 hrs) | 6204 | 6978 |
| B2 lower segment | C113 | 6/30/2012 | 4342 (2 days) | 5808 | 7046 |
| B2 upper segment | C114 | 6/4/2012 | 3560 (14 hrs) | 6049 | 7309 |

Note: Concrete Cylinder test report and Quality Control report are attached in appendix A-4 thru A-9.

In the erection drawings, CEG stated that “Grout columns and walls within 48 hours of erection unless noted otherwise (uno)”, see appendix A-14. This provision was based on the CEG’s

premise that erection would proceed at a natural pace and would only advance to one or two floors above the footing. Once the grout was placed, the erection could place the higher floors without any issues.

There was no contact or other communication between MEP and CEG during the course of the construction.

Florida Lemark Corporation

Florida Lemark had several employees at the site but just four of them were responsible for grouting and caulking. One employee was injured in the incident, and another employee left for his native country. Therefore, OSHA could only obtain information from the remaining two employees, one of whom was a supervisor. Included in the grouting scope of work were the column splice locations and the column bases above the footings. The splices were grouted with drypack, but flowable grout was used at the column bases. The procedure was to build a formwork over the footing, some 3-4" away from the column on all four sides, and then pour the grout which would then flow underneath the column. The formwork was removed the next day, and generally the grout which extended beyond the column was left as it was. Sometimes, the grout projecting beyond the column would be cut. It took a couple of hours for the grout to set.

Lemark personnel did not place the shims under the columns. The erection crew placed the shims and adjusted the leveling nuts of the anchor bolts while erecting and plumbing the columns. During interview with OSHA, Lemark said that Lemark's employees placed the grout at the columns only after the erector foreman had identified the column locations where the grout was to be placed. Lemark further said that without instructions from the erection crew, the Lemark workers would not place the grout. When asked why there were no grouts under column B3 and the A3.3 columns, they said first that they were not aware of the lack of grout, and second that if that was true, they must not have been instructed by the erection crew to place the grout at those locations. Contract documents required that grout be placed within 48 hours of the erection of columns, see appendix A-14.

Lemark's contention that it did not place the grout under the two columns because of a lack of instructions from the erection crew has little merit. First, it was in their contract that they were

responsible to grout at all locations. Second, they had been grouting the column bases at over two dozen locations after the column was set and plumbed, and before the column splice was grouted. Third, they placed grout under the column bases even before all the floors were framed with IT beams and double tees. Fourth, in the case of the B3 column, that column was standing for over two weeks, with framing completed up to the sixth floor on the west side, and still the base of the column was not grouted.

Structural Analysis and Discussion

The purpose of the structural analysis was to:

1. Determine whether the structure as designed by the structural engineer of record and the specialty engineer was designed in accord with the industry standards.
2. Determine whether the structure under construction could have supported the loads imposed upon it at the time of the incident.
3. Determine the cause of the collapse.

The structural analyses were generally limited to the area of the collapse which was bounded by column lines A to C and column lines 2 to 5. The following contract documents were reviewed.

1. Set of architectural drawings dated March 14, 2012 &/or April 25, 2012.
2. Set of structural drawings dated March 14, 2012 &/or April 25, 2012.
3. Set of erection drawings prepared by Coreslab.
4. Set of detail shop (fabrication) drawings prepared by Coreslab.
5. Set of detailed structural sheets prepared by CEG.
6. Set of calculation sheets prepared by CEG.
7. Contract specifications.

The following information provided in the general notes, pertinent to the structural analysis, are reproduced below:

1. "Florida Building Code", 2007 Edition, Project is in "High Velocity Hurricane Zone".
2. "Building Code Requirements for Reinforced Concrete", (ACI 318-05).

3. "PCI Design Handbook", 7th Edition.
 4. Minimum Design Loads for Buildings and Structures", ASCE 7-05.
 5. Parking Garage Live Load = 40 psf.
 6. Design wind load: wind speed 146 mph, Exposure C, Importance Factor = 1.15.
 7. Concrete strengths shall be as follows.
 - i. Prestressed elements $f'_c = 6,000$ psi, Minimum (u.n.o.)
 - ii. Precast elements $f'_c = 6,000$ psi, Minimum (u.n.o.)
 - iii. CIP topping $f'_c = 4,000$ psi, Minimum
 - iv. Footings $f'_c = 3,000$ psi
- Initial strength of concrete shall be a minimum of $f'_{ci} = 3,500$ psi, for prestressed members and $f'_{ci} = 2,500$ psi for precast members, unless noted otherwise.
8. Welded wire fabric shall comply with ASTM A185, black finish.
 9. Deformed bars shall comply with the requirements of ASTM A615, Grade 60, black finish.
 10. Structural plates and shapes shall be made from steel conforming to ASTM A36.
 11. Prestressing strands shall be ASTM A-416 low relaxation , 270 ksi, ½" Special strands or 0.6" diameter
 12. Column anchor bolts shall comply with the requirements of ASTM A307.
 13. Column grout shall be of Sure-Grip grout from Dayton Superior or equal, Minimum $f'_c = 8,000$ psi.

The following members were analyzed for their final condition for the finished structure.

a. Double tee

The 30" deep prestressed double tee, 12'-0" wide x 4" thick flange, was evaluated to support its dead load and superimposed garage live load of 40 psf. The double tee was supported at each end on the ledges of the IT beam. The double tee was evaluated as a simple span beam for flexure and shear. Our structural evaluation determined that the double tee reinforced with (7) 1/2"Ø strand as primary reinforcement with WWF in the flange as adequate to resist the required loads. Shear strength was also adequate.

b. Ledge design of the IT beams

The typical 24" wide x 34" deep prestressed IT beam was provided with continuous ledges 8" wide x 12" deep on either side of the IT beam. The ledge of the IT beam was evaluated for the concentrated reaction coming from the double tee. The ledge was analyzed for flexure, shear, and axial forces due to point load. Our structural evaluation determined that the WWF ($A_s = 0.135 \text{ in}^2/\text{ft}$) in the ledge and other rebars were adequate to resist the resulting forces.

c. IT beams with loads from one side and from both sides

The 34" deep prestressed IT beam in its final condition had an additional 4" thick of cast-in-place wash pour (concrete topping). The IT beam was supported at each end on the column corbel. The IT beam was evaluated for its own dead load and the load of the double tee beam from one side only. The beam was evaluated as a simple span for flexure, direct shear, and the torsion effects. Our structural evaluation determined that the (28) 0.6"Ø strands, other rebars and #4 stirrups in the IT beam were adequate to resist the required flexure and shear.

Further, the IT beam was evaluated for loads coming from both sides of the beam. Our structural evaluation determined that the (28) 0.6"Ø strands, other rebars and #4 stirrups in the IT beam were adequate to resist the required flexure and shear.

d. Spandrel beam

The 10" wide x 64" deep prestressed spandrel beam consisted of continuous 8" wide x 12" deep ledge at its bottom on interior side of the spandrel beam. The beam is supported at each end on the inside pocket of the exterior column. The beam was evaluated as a simple span for flexure, direct shear, and the torsion effects. A discrepancy was noted that the original design calculations done by CEG showed (10) ½" Ø strands and detail shop drawings drawn by Coreslab Structures showed (7) ½"Ø strands. Our structural evaluation determined that the (7) ½"Ø strands, #5 rebars and #4 stirrups in the spandrel beam were adequate to resist the required flexure and shear.

e. Wall panel, shear walls and diaphragm

The wall panels, shear walls and diaphragm design were examined, and found to be adequate.

f. Column corbel design

The IT beam was supported at each end on column corbel in the east-west direction (Fig. 5). The design of corbel was reviewed considering the reaction of dead and live loads supported by the IT beam. During our review, a discrepancy was noted between CEG's design calculation sheet and the fabrication drawing prepared by CEG for Coreslab. CEG showed 4 #4 stirrups while the Coreslab drawing showed 3 #4 stirrups. Our evaluation determined that 4 #9 main rebars and 3 #4 stirrups were adequate to resist the required flexure and shear.

g. Column

The typical interior column which was fabricated in two segments consisted of 24"x24" from the parking garage level 6 to level 2 and 42"x24" from level 2 to level 1. However, the shorter dimension of the column was oriented in the north-south direction. The two segments of the column were spliced halfway between level 3 and level 4. On both sides of the column in the east-west direction, corbels (integral part of the precast column) were provided to support the IT beams. For the splice connection of the column between two segments, see Fig. 8.

The following points were considered in the review of the column design.

- The axial load of the column was calculated based on the combination of the dead load and reduced live load of the parking garage as per the Florida Building code.
- The column was considered laterally supported at each floor level.
- The splice connection of the column was assumed to be a pin connection.
- The column was subjected to moment due to eccentricity of bearing of the IT beams. During construction, the column was subjected to eccentric loads from one side only.

- Discrepancy was noticed on the design strength of concrete between CEG's design calculations and fabrication drawings prepared by Coreslab. CEG designed the columns for $f'_c = 7,850$ psi while the column was precast using $f'_c = 6,000$ psi based on the fabrication drawings. Design of the column was reviewed using $f'_c = 6,000$ psi.
- The design of the concrete column was evaluated considering two different conditions, see below.

Evaluation 1

The column was evaluated in its final condition once the structure is completed. Our evaluation indicated that the upper segment of the column up to level 3 with 4 #11 rebars was able to resist the required loads. However, the 24"x24" column below level 3 was not able to resist the required ultimate axial loads of approximately 2,000 kips, when strength of the concrete $f'_c = 6,000$ psi was considered based on the fabrication drawings. When design strength of the concrete, $f'_c = 7,200$ psi based on the result of the specimen testing was considered, the column design was found adequate. The 24"x42" column with 8 #11 rebars below the second level was found adequate to resist the required loads.

Evaluation 2

The column was also evaluated based on the erection sequence. In this situation, only dead loads of the IT beams from one side were considered along with the cumulative eccentric moment from each floor's corbel. Our evaluation determined that the design of the precast column met the industry standards.

h. Foundation design

The foundation design was generally satisfactory. Axial forces generated at the base of the precast column are transferred to the footing by bearing on concrete through shim plate, non-shrink high-strength grout and anchor bolts. The footing design for the typical interior column consisted of cast-in-place concrete spread footing of 23'-6" x 23'-6" x 5'. The design strength of the concrete for the footing was considered to be 3,000 psi at 28

days. Our evaluation indicated that the spread footing design with 28 #11 each way on the bottom was adequate to resist the loads. The foundation was designed by the structural engineer of record. The bearing capacity of the soil was considered to be 4,500 psf.

i. The failure: Load on Column B3

As discussed earlier, the base of column B3 was not grouted. Therefore, the means to transfer the load to the footing were through the four 1¼" anchor bolts (A370), and 10x10" shim plates. Given the yield strength of the anchor bolts as 36 ksi, the maximum load that could be transferred to the footing through anchor bolts was $4 \times 36 \times 1.22 = 175$ kips. Bearing capacity over the shim plates is dependent of the strength of the concrete in the column. The following table shows a range of bearing capacity over the shim plates with different concrete strengths.

| Bearing strength of the loaded area for column B3 through 10"x10" shim plate & 4 anchor bolts | | | | | | | |
|---|----------------------------------|--------------------|--------------------------|---|--|--|--|
| Concrete Strength f'_c | Strength reduction factor ϕ | Load Factor 1.0 | Magnification factor 2.0 | Ultimate capacity [■] = $\phi \times f'_c \times 0.85 \times 2.0 \times A^{\blacktriangle}$ | Load taken by 1 ¼" ϕ bolt = 36 ksi x 1.22 in ² x 4 bolts | Total capacity to transfer the load to the footing | Actual service dead load on column at the time of incident |
| 6,000 psi per fabrication drawing | 0.65 | 1.0 | 2.0 | 663 kips | 176 kips | 839 kips | 1,360 kips greater than col. capacity |
| 7,200 [*] psi @ 28-day strength | 0.65 | 1.0 | 2.0 | 795 kips | 176 kips | 971 kips | 1,360 kips greater than col. capacity |
| 7,920 psi w/ 10% increase past 28 days | 0.65 | 1.0 | 2.0 | 874 kips | 176 kips | 1,050 kips | 1,360 kips greater than col. capacity |

■ Reference ACI 318-05, Section 10.17.1

▲ Area of 10"x10" embedded shim plate at bottom of the column

* Average of two concrete cylinder test results

We note that as stated earlier, laboratory reports of testing of concrete of B3 and other columns are not yet available to OSHA. Petrographic examination of concrete could indicate water cement ratio, presence of unhydrated cement, if any, and other flaws impacting concrete strength. However, for our analysis, we have used the concrete strength as indicated in the table.

In the above table, the actual bearing area of 100 square inches of the shim plates have been magnified by a factor of 2.0 in accordance with the applicable ACI code 318-05. The capacity reduction factor of 0.65 has been retained, also as per ACI code. Three different concrete strengths were considered. 6,000 psi was the specified strength, though the design was based upon 7,500 psi. This error occurred when CEG inadvertently indicated 6,000 as the concrete strength on the design sheet provided to Coreslab by CEG. The 7,200 psi strength was also considered because of the specimen strength at 28 days. The column was cast on June 26, 2012, and approximately 106 days had passed until the incident on October 10, 2012. It is possible that the concrete would have gained an additional strength of 10%. Therefore, a strength of 7,920 psi was also considered.

The column was erected on September 14, 2012 or earlier. On September 25, 2012 or earlier, the column was loaded with all the structural elements on the west side, with no load coming from the east, see Fig. 20. It is estimated that the total dead load at that time was approximately 730 kips based upon the actual weights of the individual pieces, not considering any load factor. As can be seen from the above table, the range of capacity of the column bearing was 839 to 1,050 kips, and thus failure did not occur.

By October 8, 2012, the second, third and fourth floor IT beams were placed, and double tees on the second, third and partially on the fourth floor were also placed, see Fig. 32. This provided an additional load of 465 kips to column B3, bringing the total load to 1,195 kips. The failure did not occur on October 8, 2012 although the total load exceeded the range of the capacity of the column bearing. The failure did not occur because either the actual concrete could have been of higher strength, or the magnification factor of 2.0 was a little conservative.

On October 10, 2012, an additional load of 165 kips was placed on column B3 because additional IT beams and double tees were placed on the fourth, fifth and sixth floors. The total load at the time of the collapse on column B3 is estimated to have been 1,360 kips, based upon the actual weights of individual pieces. The actual load exceeded the upper

value of bearing capacity by a third, and thus the failure occurred. If grout had been placed at the base of column B3, this failure would not have occurred.

| Actual dead load on column B3 (Load factor not considered) | |
|--|---------------------------------------|
| Date | Actual service dead load on column B3 |
| September 25, 2012 | 730 kips |
| October 8, 2012 | 1,195 kips |
| October 10, 2012 (Incident date) | 1,360 kips |

Seismograph

A seismograph was set up near the construction site to record and monitor ground motions due to periodic blasts in a nearby quarry which usually created perceptible vibrations inside the garage under construction. A review of the record of ground motions on the day of the incident indicated that it was of little significance to the collapse.

General Contractor

Ajax Building Corporation, general contractor, had a sizeable presence at the site including an engineer. They held weekly/daily meetings at the site and were well aware of the delivery, erection, progress, schedule and responsibilities of various contractors at the site. Ajax had direct interaction with the contractors at the site on a daily basis. Ajax had in possession all relevant construction documents including structural details prepared by CEG for Coreslab, see Fig. 9, Coreslab erection drawings, see appendix A-14, and Solar Erector erection procedures, see appendix A-16. Ajax referred to these documents during the course of their normal activities. Ajax received copies of all reports including inspection reports of the threshold inspectors. Ajax employees walked around the site a number of times daily, exposing them to the progress or lack thereof of various critical items in the construction of the project. Should a catastrophe occur, Ajax employees would therefore, be exposed. Critical items were welding of precast items to supporting members, providing temporary bracings, providing grouts at the column splices and bases among other items. Ajax received inspection reports from the threshold inspectors

regularly, and with due diligence could have known that there were eighteen columns where there were no inspection reports of the grout at the column bases. Among those eighteen columns, column B3 was erected at least 26 days before the collapse, and yet Ajax failed to take note of this critical failing. The Lemark Corporation and MEP Structural Engineering representatives were physically at the site daily, which afforded opportunities to Ajax for communication and discussion.

Coreslab Structures

Coreslab retained Florida Lemark Corporation as its subcontractor to perform grouting work at the column splices and bases of the columns. Coreslab's employee(s) frequently visited the site, inspected the erection and progress of the construction, and addressed issues brought to their attention by Solar Erectors and Lemark. For example, on October 8, 2012, when the crane struck column B2, a Coreslab engineer who was at the site, inspected the column and provided an opinion to the solar erector and others, see appendix A-2 and A-3. In the past, Coreslab would frequently advise Ajax on solutions to the erection issues faced by sub-contractors. Coreslab would also consult their specialty engineer before rendering a solution. Coreslab designed the columns, IT beams and double tees. Their engineer prepared a book of details where among other details, the grouting at the base of columns was detailed, see Fig. 9. Besides, Coreslab specifically provided a note in their erection drawings that grout be placed within 48 hours of the erection of the column, see appendix A-14.

With a pro-active role at the site, and a thorough knowledge of fabrication and structural design requirements of the components of the garage, Coreslab representative(s) by exercising due diligence would have known that the bases of the columns B3 and A3.3 had not been grouted by their sub-contractor for over 26 days in violation of the contract requirement that grout be placed within 48 hours. Coreslab could have asked their sub-contractor to take immediate remedial measures. If this action was taken even on October 8, this incident could have been averted. Coreslab had responsibility for design and fabrication of individual pieces, and placing grout at the bases of the columns was a critical part of their design. In the event of a catastrophic incident, Coreslab employee(s) would be exposed.

MEP Structural Engineering

MEP was the designated threshold inspector at the site. MEP performed poorly and in an unacceptable manner. There were three MEP employees involved in the inspection of the construction. Two were professional engineers who visited the site only when called to examine permanent welds between the structural components, or to address specific engineering issues. The rest of the inspections were left to another employee who was at the site on a daily basis. This employee was expected to inspect everything else, anchor bolts, embed items, all precast units for conformity with the contract documents, all column base connections, all grouted connections, bearing pads, etc. In regard to column bases, the contract document required MEP to “check all column base connections and all grouted connections. Spot check grout installation procedures”. The inspector was required to “immediately notify the contractor **in person** (emphasis ours), and the Architect and Structural Engineer by **telephone** (emphasis ours)” of all non-compliance with the contract documents, see appendix A-12. The inspector was expected to conduct inspections “of all precast members and all connections using both contract documents and shop drawings”, see appendix A-13.

OSHA interviewed the employee who had a daily presence at the site, and whom MEP designated to conduct all inspections other than of permanent welds. The employee displayed poor knowledge of how the precast members were assembled, its sequence or the method or means of construction. The employee said that he would examine the column splice located at a height of 30 feet from the ground from 40 ft. or more away from the column. He had no knowledge of the plumbness of the columns. He was not aware of the incident of the crane striking a column until after the incident. He said that he would complete his inspection reports based upon what he learned from other sources, and then would verify them later. He failed to write inspection reports on the base grouts of eighteen columns.

As discussed earlier, column B-3 was erected some 26 days before the incident, giving ample opportunity for the inspector to examine whether the base had been grouted. It was in plain sight of everyone that column B-3 was being loaded from the west, and then from the east. If the inspector had performed properly, he would have known that the base had not been grouted. He was then expected to inform Lemark **in person** to take immediate remedial measures. He was

also expected to contact the architect and structural engineer by telephone. If the inspector had performed properly, the incident could have been averted.

Solar Erector

Solar Erector's crew and their foreman had a daily presence at the site. Solar Erector was very conversant with the design details, in particular the column base details; see Fig. 9, prepared by Coreslab. Solar Erector had almost daily communications with Coreslab personnel. Solar Erector sought solutions from Coreslab in regard to erection issues they encountered at the site during erection of the precast members. Solar Erectors understood the critical nature of the grout at the column bases and had in their possession documents prepared by Coreslab requiring that the grout be placed within 48 hours of the erection of the column. In fact, Solar Erector's own erection procedures required that "the grouting of the column bases shall be done in a timely manner (misspelled in the original document as manor) and as soon as possible, unless noted otherwise in the Erection Drawings". It further required that "deviation from this procedure must receive approval of Precast Engineer", see appendix A-17. In spite of the details provided in Fig. 9, Coreslab requirements of 48 hours, and their own erection procedure, Solar Erector displayed plain indifference, and continued loading the B3 column with loads coming from the 2nd to the 6th floor on the west side of column B3. For 26 days, column B3 stood without grout at the base. Then on October 8 and 10, Solar Erectors began placing loads from the east side of column B3 until the incident occurred. With reasonable diligence, Solar Erectors would have known that Lemark had not grouted B3 column, and could have taken immediate remedial measures. Solar Erector failed to provide adequate support to the structural frame in violation of OSHA standard 1926.704(a).

Florida Lemark Corporation

Coreslab contracted with Lemark to perform all grouting work at the construction site for the parking garage. Lemark's crew consisted of seven or more employees at the site on a daily basis, but only four of them were responsible for grouting. Lemark knew that they were the only contractor at the site performing grouting at the bases and splices of the columns. In fact, they had already grouted a number of column bases and splices. Column B3 was standing for

approximately 26 days without grout at its base, and was in plain sight of everyone at the site. Lemark should have grouted the base within days of the erection of the column, but did not. Another opportunity was afforded to Lemark to realize that it had failed to grout the base of column B-3 when Lemark grouted the splice of the same column, some 30 ft. above the base. With reasonable diligence, Lemark would have easily observed that the base of column B3 had not been grouted, and thus could have taken immediate action. The base of the column with or without grout does not look the same because of 2 or 3 inches of projection of the grout beyond the perimeter of the column. To trained employees such as Lemark's employees, it would have been very obvious with a minimum amount of attention that the base of column B3 had not been grouted.

Conclusions

1. The Miami Dade College Garage collapsed during construction because grout was not placed, as required, at the base of an interior column to adequately transfer the column load to the footing. As loads on the column gradually increased on the day of the incident, the bearing of the column over the shim plates exceeded its capacity, resulting in failure. This triggered a cascade of collapse of columns, inverted tee beams and double tees on all five floors weighing approximately 3,300 tons over an area of approximately 16,000 square feet.
2. The structural engineering company responsible for threshold inspection performed improperly by not having checked the grout at the bases of the columns at approximately a dozen locations. The threshold inspection company's contention that they did not inspect the grout because they were not asked to do so by the contractor has little merit. The general duty clause, section 5(a)(1), of the OSH Act was violated.
3. The contractor responsible for placing the grout at the bases of the columns was negligent in that at least one interior and one exterior column were not grouted. The contractor had adequate time to place the grout as the columns were erected over approximately 26 days before the collapse. The construction document required that grout be placed within 48 hours of the erection of the column. The contractor's argument that he was not specifically asked by the erector to place the grout under the columns is flawed. The general duty clause, section 5(a)(1), of the OSH Act was violated.
4. The structural design of the garage generally met the applicable design standards. The specialty engineer inadvertently indicated 6,000 psi as the concrete strength of a certain column instead of 7,500 psi. This did not contribute to the collapse.
5. The precast erector failed to brace the columns in the north-south direction on the day of the incident. The precast erector did not adequately support the B3 column, thereby causing collapse of the column. The precast erector thereby failed to maintain structural stability during construction. OSHA standard 1926.704(a) was violated.

6. The erector did not comply with the contract requirement that “welding of connections shall follow closely behind the erection of units”. The double tees were not welded to the IT beams between column B2 and B3 on the fifth and fourth floors, contributing to instability.
7. For approximately 26 days, the bases of columns B3 and A3.3 were not grouted. By exercising due diligence, the general contractor who had control and authority over the site, and direct access to inspection and progress reports, could have known that these column bases had not been grouted, and could have asked for immediate remedial measures. The general duty clause, section 5(a)(1), of the OSH Act was violated.
8. Coreslab representatives were frequently present at the construction site providing solutions to issues faced by the precast erector and others during erection of the garage. By exercising due diligence, Coreslab could have known that the bases of at least two columns B3 and A3.5 had not been grouted by the sub-contractor retained by them, and could have asked their sub-contractor to take immediate action. The general duty clause, section 5(a)(1), of the OSH Act was violated.

APPENDIX

CORESLAB[®]
STRUCTURES (MIAMI) INC.

October 12, 2012

Mr. Rob Culpepper, Sr. Project Manager

Ajax Corporation

Ref: MDC Parking Garage

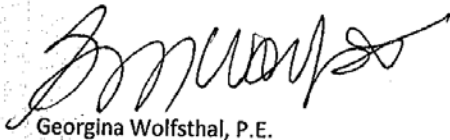
Coreslab Project 1203

Dear Sir:

Attached please find the site visit of our Project Manager with regard to the Column at B-2. It had been reported that the column had been bumped by the crane. Susan Perez is an engineer and our qualified representative in the field. She made a visual observation of the column and no visible damage was seen. Based on this observation, no further investigation was warranted and erection could proceed.

Please contact me if you have any questions.

Sincerely,



Georgina Wolfsthal, P.E.

Engineering Manager

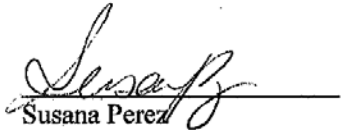
Coreslab Structures (Miami), Inc.

10501 N.W. 121ST. WAY • MEDLEY, FLORIDA 33178
(305) 823-8950 • FAX (305) 825-8457

ARIZONA • ARKANSAS • CALIFORNIA • CONNECTICUT • FLORIDA • GEORGIA • OKLAHOMA • ONTARIO • SOUTH CAROLINA • TEXAS

Project Name: MDC Parking Structure
Job No. 1203
Site Visit
10/10/12

I was made aware of the column bump by Manuel Rodriguez, and attended the meeting at Ajax on Tuesday afternoon, where it was discussed. I observed the column on Wednesday morning with Bob White of Solar Erectors, and John Rincon, of Ajax Construction. Only a dark scuff mark was visible on the North side below the column splice. The column was loaded with product at the time. No cracks, dents, spalls, or chips were visible to me.



Susana Perez
Coreslab Structures (Miami), Inc.

:File

Coreslab Structures (Miami), Inc.


Quality Control Reports
Miami Dade College Parking Garage

Issued To: Ajax Building Corporation

J. Orlando Rivera

Project 1203

Prestress Reports, From 6/29/12 date to 7/20/12 date.

| Date cast: | Mark Nos. | Strength Reqd | Strength Ach'ved | Approval |
|--------------------------------|------------------------|---------------|------------------|--|
| Bed # | Line # | at 28 days | at 28 days | |
| WP99,ST40,SW12,LW2,LW48 | | 6000 | 7006 |  |
| C34,C32,SP21,ST27 | | 6000 | 7396 | |
| 6/26/12 | | | | |
| T-5N | T-195 (2),268,269,138, | 6000 | 8002 | |
| | T-138,281 | | | |
| | 127 | | | |
| T-5S | same | 6000 | 8483 | |
| | 127 | | | |
| B-24 | B-19(2),20 | 7000 | 7546 | |
| | 21 | | | |
| B-12 | B-106,108,107 | 6000 | 7026 | |
| | 201 | | | |
| ST36, LW23,SP31,ST24,SP22,LW48 | | 6000 | 7062 | |
| C111,SP72,C9,WP122 | | 6000 | 7081 | |

QC report showing B3 lower segment C111

Coreslab Structures (Miami), Inc.

Quality Control Reports
Miami Dade College Parking Garage

Issued To: Ajax Building Corporation

J. Orlando Rivera

Project 1203

Prestress Reports, From 7/06/12 date to 7/20/12 date.

| Date cast: | Mark Nos. | Strength Req'd | Strength Ach'ved | Approval | |
|------------|--------------------------------|----------------|------------------|----------|--|
| Bed # | Line # | at 28 days | at 28 days | | |
| 6/29/12 | | | | | |
| T-1S | T-266(2),268,150,113 | 6000 | 8012 | | |
| | T-114,293,140 | | | | |
| | 26 | | | | |
| T-1N | above | 6000 | 8260 | | |
| | 26 | | | | |
| | ST46,ST28,ST62,WP108,LW91,SP75 | 6000 | 7428 | | |
| | LW27,SP32,LW53 | 6000 | 6248 | | |
| 6/30/12 | | | | | |
| T-5N | T-196,152,269,138,141,266,195 | 6000 | 8126 | | |
| | 129 | | | | |
| T-5S | above | 6000 | 8499 | | |
| | 129 | | | | |
| | ST4,SW12,LW58,WP123 | 6000 | 7452 | | |
| | C113,SP33,ST37 | 6000 | 7046 | | |

QC report showing B2 lower segment C113

CORCON LAB

STRUCTURES (MIAMI) INC

QUALITY CONTROL DEPARTMENT

DAILY CONCRETE CYLINDER TEST REPORT

| JOB # | CYL TEST | CHAN COMP | MARK # | TIME | | QTY CYL | AIR TEST | SLUMP | SPREAD | VSI SEC | TEMP | MIX CODE | STRENGTH SPECIFIED | 1ST ACTUAL READING | | | 2ND ACTUAL READING | | | 3RD ACTUAL READING | | | PRECAST | |
|--------------------|----------|-----------|--------------|--|---------|---------|----------|---------------|--------|--------------|------|------------------------------------|--------------------|----------------------------|-----|------------------------------|--------------------|-----|-----|--------------------|-----|------|---------|-----|
| | | | | POURED | BROKEN | | | | | | | | | PSI | HRS | PSI | HRS | PSI | HRS | PSI | HRS | PSI | HRS | PSI |
| DATE CAST: 6-30-12 | | | | WEATHER: <input checked="" type="checkbox"/> SUNNY <input type="checkbox"/> WINDY <input type="checkbox"/> CLOUDY <input type="checkbox"/> RAINY <input type="checkbox"/> LOW 81 | | | | TEMP. HIGH 93 | | TEMP. LOW 81 | | | | TEST DATE (7 DAYS): 7-7-12 | | TEST DATE (28 DAYS): 7-28-12 | | | | | | | | |
| 1113 | A | | MP-4 | 10:40 AM | 4:30 AM | 4 | 1.1% | 8" | | | 81° | 018012 R 4.4-5.1 | 3000 | 4268 | 2-D | | | | | | | 5102 | 610 | |
| " | " | | P-24 1/2 | | | " | " | " | | | " | " | 3000 | | | | | | | | | | 6384 | |
| " | " | | P-58 1/2 | | | " | " | " | | | " | " | 2500 | | | | | | | | | | | |
| " | " | | P-37 | | | " | " | " | | | " | " | 2600 | | | | | | | | | | | |
| " | " | | P-15 | | | " | " | " | | | " | " | 3000 | | | | | | | | | | | |
| " | " | | P-81 | | | " | " | " | | | " | " | 3000 | | | | | | | | | | | |
| " | " | | P-41 | | | " | " | " | | | " | " | 3000 | | | | | | | | | | | |
| " | " | | P-19 | | | " | " | " | | | " | " | 3000 | | | | | | | | | | | |
| " | " | | P-56 | | | " | " | " | | | " | " | 3000 | | | | | | | | | | | |
| 1203 | B | | SI-47 | 1:25 PM | 4:35 AM | 4 | 2.5% | | 26" | 0.5 | 91° | 775 SC 3.1-57.0.9-A1 1.0-R-1 | 2500 | 4565 | 2-D | | | | | | | 6030 | 7594 | |
| " | " | | LW-58 | | | " | " | " | | 4.5 | " | " | 2500 | | | | | | | | | | 7452 | |
| " | " | | WP-123 | | | " | " | " | | " | " | " | 2500 | | | | | | | | | | | |
| " | " | | SI-4 | | | " | " | " | | " | " | " | 2500 | | | | | | | | | | | |
| 1203 | C | | ST-37 | 2:55 PM | 4:40 AM | 4 | 2.6% | | 28" | 1.0 | " | 775 SC 3.1-57.0.9-A1 1.9-R-1 | 2500 | 4342 | 2-D | | | | | | | 5808 | 7046 | |
| " | " | | SP-22 | | | " | " | " | | 3.5 | " | " | 3500 | | | | | | | | | | 7228 | |
| " | " | | C-113 | | | " | " | " | | " | " | " | 2500 | | | | | | | | | | | |
| 1207 | D | | willy sample | 11:30 AM | 4:45 AM | 4 | 1.2% | 9 3/4" | | | | 060812 | N/A | 4311 | 2-D | | | | | | | 5311 | 7612 | |
| " | " | | | | | " | " | " | | | | | | | | | | | | | | | 7388 | |

Daily Concrete Cylinder Test Report – B2 lower segment C113

6/04/12

| | | | |
|---------------------------|-------------------------------|------|------|
| T-1N | T-254 (4),177,137,180,180,243 | 6000 | 8469 |
| | 16 | | |
| T-1S | same | 6000 | 8333 |
| | 16 | | |
| SW19, S201,SW3,LW38, B202 | | 6000 | 7785 |
| C8, C114,SP28,C26,LW39 | | 6000 | 7309 |

6-05-12

| | | | |
|-------------------------|------------------|------|------|
| T-5S | T-165,257(3),149 | | |
| | 147,136,213 | 6000 | 8791 |
| | 119 | | |
| T-5N | same | 6000 | 8455 |
| | 119 | | |
| B-24 | B12,3,9,14,1 | 7000 | 7094 |
| | 13 | | |
| ST17,LW34,WP37,WP88,SW4 | | 6000 | 7480 |
| C24,SP29,C135,C26,LW73 | | 6000 | 7388 |

6/06/12

| | | | |
|------|-------------------------------|------|------|
| T-1N | T-257 (4),178,180,180,243,243 | 6000 | 8514 |
| | 17 | | |
| T-1S | same | 6000 | 8154 |
| | 17 | | |

QC report showing B2 upper segment C114

CORCON SLAB STRUCTURES (MIAMI) INC
QUALITY CONTROL DEPARTMENT
DAILY CONCRETE CYLINDER TEST REPORT

| JOB # | DATE CAST: MON. TUE. WED. THUR. FRI. SAT. | CHAN / COMP | MARK # | WEATHER: | | | SLUMP | SPREAD | VSL | TEMP | MIX CODE | STRENGTH SPECIFIED | PRECAST | | | TEST DATE (7 DAYS) | TEST DATE (28 DAYS) |
|-------|---|-------------|--------|----------|--------|-------|-------|--------|------|--------------|----------|--------------------|--------------------|--------------------|--------------------|--------------------|---------------------|
| | | | | POURED | BROKEN | TIME | | | | | | | 1ST ACTUAL READING | 2ND ACTUAL READING | 3RD ACTUAL READING | | |
| | | | | TIME | QTY | WINDY | | | | | | | | | | | |
| | | | | POURED | CYL | CL | LOW | SEC | | | | | | | | | |
| 1203 | 6-4-12 | A-25 | SW-17 | 12:15 PM | 5 | 3.3 | 24" | 45 | 77.5 | SC | 3000 | 3931 | PS1 | 6210 | 6-11-12 | 7-2-12 | 7851 |
| | | | SW-21 | 4:30 AM | 5 | 3.3 | | | 46 | 4.6-5/1.0-A7 | 3500 | 10/4 | HRS | | | | 7785 |
| | | | B-202 | | | | | | | | 2500 | | PS1 | | | | |
| | | | SW-3 | | | | | | | | 3000 | | HRS | | | | |
| | | | LW-38 | | | | | | | | 2500 | | PS1 | | | | |
| | | | | | | | | | | | | | HRS | | | | |
| 1203 | | B-26 | C-6 | 2:35 PM | 5 | 2.5 | 20" | 1.0 | 90 | 77.5 | SC | 2500 | PS1 | 6049 | | | 7587 |
| | | | C-114 | 4:35 AM | 5 | 2.5 | | 35 | 46 | 4.6-5/1.0-A7 | 2500 | 14 | HRS | | | | 7309 |
| | | | SW-28 | | | | | | | | 3500 | | PS1 | | | | |
| | | | C-26 | | | | | | | | 2500 | | HRS | | | | |
| | | | LW-39 | | | | | | | | | | PS1 | | | | |
| | | | | | | | | | | | | | HRS | | | | |
| | | | | | | | | | | | | | PS1 | | | | |
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| | | | | | | | | | | | | | HRS | | | | |
| | | | | | | | | | | | | | PS1 | | | | |
| | | | | | | | | | | | | | HRS | | | | |

Daily Concrete Cylinder Test Report – B2 upper segment C114

MEP STRUCTURAL

INSPECTION REPORT

Structural-Building-Mechanical-Electrical-Plumbing-Fire Protection

| | | | |
|------------|---------------------------------------|--------------------|--|
| DATE | <u>08/07/2012</u> | REPORT No. | <u>1082</u> |
| PROJECT | <u>MDC West Campus Parking Garage</u> | MEPS FILE No. | <u>900.043</u> |
| LOCATION | <u>Doral, Fl</u> | PERMIT No. | <u>2012-006-B Foundation</u> |
| OWNER | <u>Miami Dade College</u> | CLIENT PROJECT No. | <u>SOR-11001</u> |
| CONTRACTOR | <u>AJAX Building Corporation</u> | INSPECTOR | <u>H.Vergara, P.E. Eduardo Martin, BN5510.</u> |

INSPECTION TYPE: STRUCTURAL BUILDING MECHANICAL ELECTRICAL PLUMBING FIRE
 UNDERGROUND ROUGH PARTIAL IN-PROGRESS RE-INSPECTION FINAL

MEMBER / AREA(s) INSPECTED: Inspection Code(s): **(110) Precast Joist/Beams**
 Building(s) / Area(s):
 Room(s): Floor(s) / Elevation(s):
 Column Line(s): Grid Line(s):

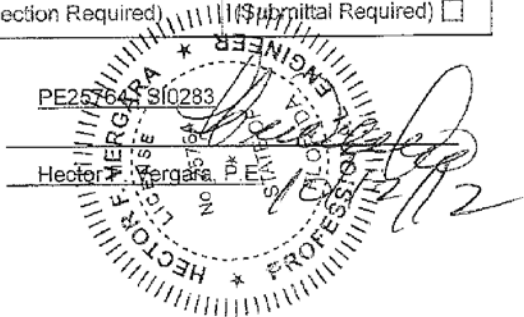
COMMENTS:
 Inspected the erection of precast elements. Spot check of bolt tightening was conducted and visual inspections to all bolts. Shims, washers, bearing pads, nuts and grout between elements were used as per project specifications. Final welding and grout will be inspected on a later day. Elements inspected.
 Columns – C16, 10, 112, 205, 3. Beams – B3, 21, 23, 21. Stair walls – ST26, 25; SW21, 21, 13, 14, 15. D Tees – T239, 239, 239, 239, 239, 239, 239, 239, 216, 107, 239, 263, 263, 284, 264, 279, 239, 263, 284, 217, 104, 105, 132, 134, 106. Spandrels – SP25, 25, 87, 83, 77, 29, 28; Slabs S201.

ITEMS OF NON-COMPLIANCE REQUIRING RESOLUTION: Need approved set of plans and shop drawings for prefab. Concrete panels by building Department concluding field modification detail for anchor bolts.

DISPOSITION OF INSPECTION: APPROVED REJECTED (Re-Inspection Required) PENDING (Submittal Required)

DISTRIBUTION:
 OWNER BUILDING DEPT.
 CONTRACTOR ARCHITECT
 ENGINEER FILE
 OTHER Judy Gonzalez

LICENSE No. PE25764 S10283
 SIGNATURE Hector Vergara, P.E.



COPY

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 www.mepstructural.com

MEP Report - Inspection of column B3 upper segment C112

MEP STRUCTURAL

INSPECTION REPORT

Structural-Building-Mechanical-Electrical-Plumbing-Fire Protection

| | | | |
|------------|----------------------------------|--------------------|--|
| DATE | <u>10/6/2012</u> | REPORT No. | <u>1233</u> |
| PROJECT | <u>MDC West Parking Garage</u> | MEPS FILE No. | <u>900.043</u> |
| LOCATION | <u>Doral, Fl</u> | PERMIT No. | <u>2012-006-B</u> |
| OWNER | <u>Miami Dade College</u> | CLIENT PROJECT No. | <u>SOR11001</u> |
| CONTRACTOR | <u>AJAX Building Corporation</u> | INSPECTOR | <u>H.Vergara, P.E., Martin, BN5510</u> |

INSPECTION TYPE: STRUCTURAL BUILDING MECHANICAL ELECTRICAL PLUMBING FIRE
 UNDERGROUND ROUGH PARTIAL IN-PROGRESS RE-INSPECTION FINAL

MEMBER / AREA(s) INSPECTED: Inspection Code(s): 110
 Building(s) / Area(s):
 Room(s): Floor(s) / Elevation(s):
 Column Line(s): Grid Line(s):

COMMENTS: Inspected the erection of precast panels.
 -Spot check on bolt tightening was conducted and visual inspection on all bolts.
 -Shims, washers, nuts, bearing pads and grout between elements as per project specifications. Final weldings and grout will be inspected on a later date. Elements inspected:
 Column C-113-14 / Beams B-5,18 / Spandrel SP-7 Double Tees T138,119,120,106,128,139,138,142,120,121

ITEMS OF NON-COMPLIANCE REQUIRING RESOLUTION: 1-Pending approval or field modification detail from project design engineer for the 4" gap between the existing building and the new garage connections including the L Beam to haunch connection.
 2-Pending comments from project design engineer about the time of placement of more than 90 minutes for the secondary pour for wash and core steel at 6th, 5th, 4th and 3rd floor from line 11 to 8.
 3-Repair detail for Double Tee Stern damaged in 5th floor by column line 5 between grid lines C and D.

DISPOSITION OF INSPECTION: APPROVED REJECTED (Re-Inspection Required) PENDING (Submittal Required)

DISTRIBUTION:
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 ENGINEER FILE
 OTHER Judy Gonzalez

LICENSE No. S.I.0283, PE25764

SIGNATURE Hector F. Vergara, P.E.



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MEP Report - Inspection of column B2 lower segment C113 and upper segment C114

1.4 REPORTING

- A. The Special Inspector shall record progress, working conditions, observations, testing, deviations from the Contract Documents, and any required corrective action. He shall retain the records for a minimum of 7 years after completion of the project.
- B. The Special Inspector shall immediately notify the Contractor in person, and the Architect and Structural Engineer by telephone, of materials, tests, equipment, workmanship or construction that:
1. Does not conform to the Contract Documents, or
 2. Is not inspected or tested and cannot be inspected or tested in place.
 3. The Special Inspector shall then immediately issue those exceptions in writing to those listed above and attach a copy to the daily Inspection Report.
- C. The Special Inspector shall keep an exceptions file and review it on a daily basis, updating as exceptions are rectified. If any exceptions are not resolved in a timely manner, the Special Inspector shall issue a non-compliance notice to the Contractor and shall copy the Enforcing Agency, Owner's representative, Architect and Structural Engineer.
- D. After each inspection, the inspector shall write and sign an Inspection Report. The Report shall include the following:
1. The name and location of Project; name of Inspector; Permit Number; date; working conditions, including weather and temperature; and type and location of work being performed.
 2. Details of each inspection, including the presence and activities of the Testing Agency.
 3. Note deficiencies in the work and any unusual circumstances affecting the performance of work, including changes in materials or work sequence. Place emphasis on recurring deficiencies.
 4. Identify corrections to deficiencies listed in previous reports.

2.4 STRUCTURAL PRECAST CONCRETE:

- A. The Contract Documents place responsibility for plant and field inspection, testing, and temporary bracing with the Fabricator's Quality Control Program. The Special Inspector shall become familiar with this Program.
- B. The Special Inspector shall inspect all precast members and connections using both the Contract Documents and the Shop Drawings.
- C. In-plant inspections: visit each fabrication shop at the start of each different piece type and verify that, in general, the Fabricator is complying with all aspects of the Quality Control Program and the Contract Documents. Spot check specific conditions. Issue a report of all findings.
 - 1. Identify, examine and review plant testing of materials, subassemblies and their anchorage, plant Quality Control Program and personnel. Verify that all welders are AWS Certified.
 - 2. Spot check configuration of members; number, size and position of tendons; reinforcing steel; openings; blockouts; embedded plates; and other incorporated materials,
 - 3. Spot check conveying, placing, consolidating, finishing and curing of concrete.
 - 4. Obtain Certified Tensioning Records to verify proper tensioning.
 - 5. Generally observe plant equipment, working conditions, weather and other items that might affect the product.
 - 6. As a guide, follow the record keeping requirements of PCI MNL 116, Chapter 6.2.
- D. Field inspection: provide all inspections required by the Contract Documents. Complete all inspections and verify compliance prior to concealment.
 - 1. Review the Delegated Engineer's Bi-weekly Summary Report of inspection and testing activities and report any deficiencies. Verify that the Delegated Engineer visits the site as required by the Contract Documents.
 - 2. Inspect setting of anchor bolts, embeds and other miscellaneous structural items for size, quantity and finish.
 - 3. Check steel as received for possible damage during shipping.
 - 4. Verify that precast units are properly located in the structure by confirming that the Mark Number matches that shown on the Shop Drawings. Check that erection sequence and all permanent bracing and supports are in accordance with Approved Submittals.
 - 5. Confirm that member length, depth, width, camber and side bow are within allowable tolerances. Verify that bearing conditions comply with specified requirements.
 - 6. Inspect all field connections and verify connection material, sizes and configurations for embeds and connectors.
 - a. Visually examine all field welds and spot check all shop welds for type, size, length and quality. Verify that specified testing is performed by the Testing Agency. Verify that welds are clean and free from slag and that rust protection, if required, has been applied as per specifications. Verify that all welders are AWS Certified.
 - b. Check all column base connections and all grouted connections. Spot check grout installation procedures.
 - c. Verify bolt type, size and quantity in all bolted connections. Check that bolts are clean and lubricated, have proper washers, and conform to the Specifications. Check that bolt holes are the specified type and size. Visually verify proper degree of bolt tightening.
 - d. Inspect all threaded couplers.
 - e. Verify bearing pad material, size, position and flushness with adjacent materials.

From Project Drawing S1.02 - Part II INSPECTIONS

3.0 ERECTOR

- 3.1 The components shall be erected in accordance with these drawings and within PCI recommended tolerances. All notes to the erector, shall be carefully reviewed and followed. All units shall be set to the dimensions shown on these drawings. The Erector shall shim adjacent deck members as required to match any camber differential and any other adjacent units, if required for alignment.
- 3.2 WELDING: All welds shall be made by welders who have been certified by AWS (for the types of weld required for this project) within the last 12 months. All exposed welds shall be cleaned and coated with two coats of cold galvanizing compound. Welding of connections shall follow closely behind the erection of the units. Special care must be taken in the flange to flange connections of pretopped double tees. Connections shall not be overheated during welding which may cause cracking or spalling of adjacent concrete. All cracked or spalled concrete shall be thoroughly removed and replaced. All welding shall be E70xx electrodes CE7013, E7016, E7018 suitable for welding hot-dip galvanized material.
- 3.3 PATCHING: Coreslab shall patch recessed connections and erection inserts as specified on these drawings. The erector shall patch all erection inserts, unless noted otherwise on these drawings. Repairs may be made with the authorization of the Architect/Engineer. The patching material shall be non-shrink, non-metallic grout and shall match the color and texture of adjacent concrete as required.
- 3.4 GROUT: Dry pack under column base plates, bolt pockets and load bearing wall panels. Grout shall be mixed and applied in accordance with the manufacturer's recommendation. Grout columns & walls within 48 hrs. of erection unless noted otherwise (u.n.o.).
- 3.5 BEARING PADS: Beam, double tee bearing and connection pads shall be placed at



Solar Erectors U. S. Inc.

ERECTION PROCEDURES

- 11) Handling of pieces during erection should follow industry standard methods, per the PCI Erectors Manual unless specific instructions are shown on the drawings. Specific instructions on the drawings will supersede the minimum standards.

COLUMNS

- 1) Columns left free standing at the end of the workday shall be temporarily braced in two directions until the column is connected to the structure or grouted.
- 2) Columns should be plumb and set using the bearing haunch elevations as reference points.
- 3) Column lifting loops are to be cut ONLY after the column and the supported levels are in place.
- 4) The grouting of the column bases shall be done in a timely manor and as soon as possible, unless noted otherwise in the Erection Drawings. Deviation from this procedure must receive approval of Precast Engineer.
- 5) Grout shall be mixed and placed according to the manufacturer's instructions. Grout shall be placed by forming and pouring using a flowable mix. Grout shall be placed from one side and form shall be at least ¼" higher than base plate. Care must be made to remove all air pockets.
- 6) Base plate and/or anchor bolt repairs shall be made per the engineer's recommendation. Repairs must be approved by the Precast Engineer prior to performing the work.

BEAMS/SPANDRELS

- 1) Members shall be handled at the lifting points provided.
- 2) Beam to column, and spandrel to column, connections shall be completed before the beams are loaded unless otherwise noted.
- 3) Pads shall be placed as shown on the individual erection details of the Erection Drawings. The pads are to be placed to provide the maximum bearing area, within the constraints of 1.9 above.

DOUBLE TEES

- 1) Double tees are to be placed with equal length bearing at both ends. In no case shall the bearing provided be less than 3". Any condition providing less bearing shall be brought to the Precast Engineer's attention for review before final connections, to adjacent members, are made.
- 2) Double tees should be shimmed and leveled as they are set.
- 3) A minimum of three double tee flange connectors, and one end connection (from double tee to support member, each end) should be welded before the