PRECAST DOUBLE TEE ROOF COLLAPSE INVESTIGATION - A CASE STUDY

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ABSTRACT

During heavy rains in 2009, a portion of a concrete double tee (DT) roof system of an industrial building collapsed. Subsequently, an investigation ensued in order to determine the cause, and if other DT members were damaged. Remaining DT members with deflections between one and three-and-a-half inches, under dead loads, were discovered and the authors were tasked with determining the cause of the deflection as others had opined the deflections were due to long-term creep. This paper outlines the investigative process including historical loading events/collapse, weather research, and methodology used to determine the cause of the large deflections. Through investigation, it was determined that the collapse was due to significant rainfall that occurred in a short time, coupled with an inadequate roof drainage system and poor maintenance of roof drains. Calculations showed that the excessive deflections were not due to long-term creep but to a one time loading event that caused the permanent strain set in the DT strands. Although prestress/precast concrete structures are resilient, this case study highlights the importance of proper design and maintenance. Architects, engineers, building owners, and maintenance personnel can all learn lessons through this case study.

Keywords: Maintenance, rain loads, collapse, progressive ponding deflection

INTRODUCTION

LOCATION AND CONSTRUCTION

The subject structure is an approximately 96,000 square foot building located in the greater Denver area. An aerial view of the building is shown in Figure 1 wherein the collapsed area can be seen. The building was constructed in 1977 and is currently utilized as a warehouse and manufacturing facility. The exterior walls are partially grouted reinforced concrete masonry walls supported on spread footings. The roof structure consists of simply supported precast prestressed concrete double-tee (DT) sections that are eight feet wide and seventy five feet long with two 24-inch-deep stems. At the perimeter of the building, the DTs are supported by the concrete masonry and at interior locations the DTs are supported by precast prestressed inverted tee beams that bear on precast concrete columns. The roof can be divided into four bays with each bay being seventy feet wide corresponding with the span of the DT members.

Fig. 1 Aerial view of building

The DT members have no concrete topping slab and are connected together with field-welded embedded steel plates. At 1/16 inch per foot, the roof of the structure has minimal slope and does not meet today's slope requirements of 1/4 inch per foot. The roofing system consists of an original tar and gravel system that was overtopped with 1/2 inch gypsum board and a thermoplastic (TPO) roof membrane in 2003. All of the roof drainage flows to the south end of the roof where interior roof drains and emergency overflow scuppers are located. During the 2003 re-roof, one overflow scupper, located in the parapet wall, was covered over by the new roofing membrane.

PRIOR EVENTS

During heavy rains in 1990, a portion of the DT roof structure in Bay 2 collapsed. The engineer of record for the 1990 repair and replacement of double-tees made notes on the structural plans requiring that the roof drainage system be analyzed and upgraded if necessary. Subsequent to the 1990 collapse, two new roof drains were added but then one of the overflow scuppers was covered in 2003.

MAINTENANCE

Photographs taken soon after the 2009 collapse showed that one roof drain was completely covered by debris and another roof drain was partially covered by debris indicating that the roof drains were not well maintained prior to the 2009 collapse

COLLAPSE

On June 24, 2009 a severe thunderstorm impacted the building resulting in collapse of a portion of the DT roof structure in Bay 1 as shown in Figure 2. Fortunately, this collapse happened after normal business hours otherwise the collapse may have resulted in loss of life as workers often occupied the area below the collapse.

Fig. 2 Collapsed double tee in Bay 1

WEATHER HISTORY

A meteorologist was employed in order to understand the magnitude of the June 2009 rain event. The total rainfall of more than two inches was not as impressive as how rapidly the rain fell. The meteorologist determined that two inches of rain fell in less than 60 minutes which is unusual for the Denver area. The meteorologist also determined the storm had a minimum 100-year mean return interval. Another interesting finding was that at the beginning of the storm, small diameter hail fell. Given the size of the roof (96,000 square feet), the limited number of roof drains (nine), and observations in similar storms, it was concluded that the small diameter hail likely washed to the south end of the roof where it likely restricted flow into the roof drains.

CAUSE OF COLLAPSE

It did not take long to determine that the collapse was caused by the severe rainfall and that the covered-up overflow scupper, limited roof slope, new roof membrane, hail accumulation, and ill-maintained drains were all contributing factors to the collapse. The rainfall runoff accumulation at the south end of the roof reached a depth great enough to catastrophically fail the DTs in Bay 1. One might assume that the depth of water would have been consistent across the entire south end of the roof and this would be true to an extent. However, given that each bay acts independent (structurally speaking) and that each bay had its own unique configuration of drains (or lack thereof) it is likely that the depth of water somewhat varied between bays leading to the possibility of progressive ponding and a likely explanation as to why Bay 1 collapsed while other nearby Bays sustained no damage. Further, once the collapse occurred, the hole in the roof served as a drain mechanism for the roof to rapidly drain the other bays.

INVESTIGATION

DEFLECTIONS

After the collapse, the health of the remaining DTs was called into question. Were the adjacent and original 1977 DTs compromised? Were the newer 1990 DTs damaged? In order to determine the health of the remaining DTs, the deflection under dead loads was measured for each DT. Figure 3 shows the measured deflections at each DT wherein a positive (+) symbol depicts camber and a negative (-) symbol depicts downward deflection. Unexpected deflections were found at the south end of the building and at this point an investigation began in order to determine the cause of the large deflections since opposing experts opined the cause was long-term creep. Localized low areas at the south end of the roof were found to pond to two inches of water and questions arose as to whether or not standing water could have induced the observed deflections due to accelerated creep.

Most of the DTs at the south end of the building were found to have no camber and excessive downward deflections under self-weight and a roof superimposed dead load of 7.7 pounds per square foot (psf). Most surprising was the finding that the 1990 DTs, manufactured with low-lax strands and three inches of theoretical initial camber, also had up to two inches of deflection under roof superimposed dead load. Other, 1977 DTs on the south end of Bay 1 had deflections of up to 3.5 inches under self-weight and roof superimposed dead load alone.

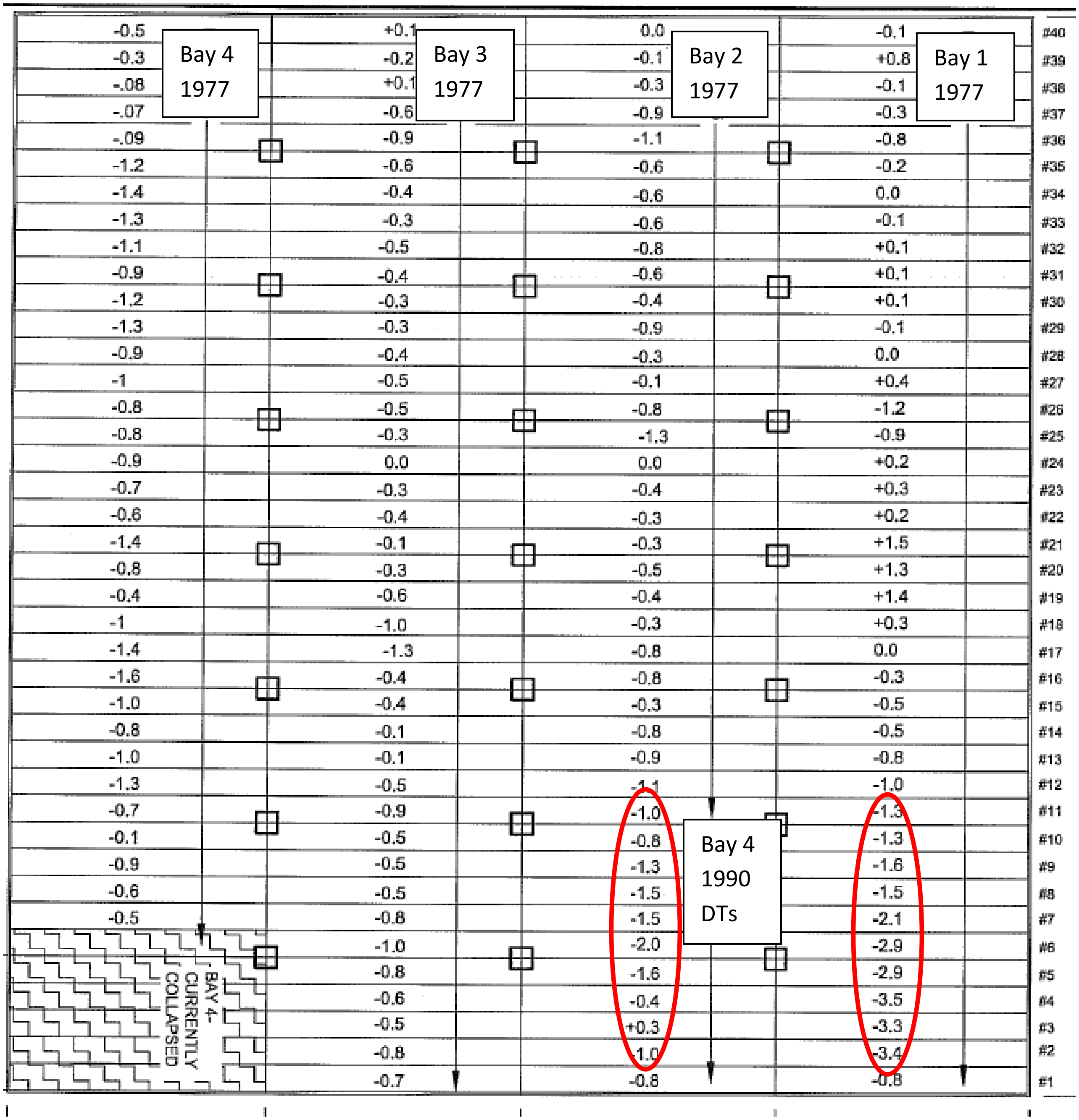

Fig. 3 DT deflections (north is up)

Also of interest was the finding that most of the DTs throughout the building had little-to-no camber but yet in rare occasions some had camber of up to 1.5 inches. Calculations from a rational analysis estimated that the DTs were likely manufactured with 2.60 inches of initial camber, (camber at erection of the precast members). This estimate is consistent with industry practice for that time. In addition, the analysis estimates that downward deflection due to the roof superimposed dead load was approximately 0.85 inches. There was no history of overload or large long-term dead loading on the remainder of the roof so it was concluded that the original DTs had a loss of prestress resulting in lost camber. However, the analysis of the original DTs only predicts a loss of 0.60 inches of camber, due to long-term creep, as opposed to the remaining camber (after dead load deflections) of 1.75 inches. Designers who have occasion to perform calculations on older DTs should take note of this finding - older DTs may not behave as predicted.

Based on the collapse history and the significant loss of prestress in certain DTs, the authors told the building owner it was not safe to occupy the building during major rain or snow events. A structural monitoring system (consisting of a string and plumb bob which would lie down on the floor under large loads) was recommended so that personal could be educated to recognize excessive deflections in the DTs until repairs could be completed.

The deflections of the DTs that ran parallel and immediately adjacent to the south wall were not as severe as other DT members in the first bay north of the south wall. This is attributed to steel diaphragm connections between the DTs and the south CMU wall that provided added stiffness to those DTs. Thus, DTs throughout the building lost camber but the DTs furthest south did not lose as much camber and this, coupled with the 1/16 inch per foot slope, resulted in a condition that naturally created low areas in the roof, which collected water.

VISUAL INSPECTION

During a visual inspection of the deflected DTs, no flexural or shear cracking was observed. An inspection of the ends of the DTs indicated that strand slippage did not occur. Observations also confirmed that no that large loads had been placed on the underside of the DTs to cause excessive deflections.

1977 DTs

For the 1977 DTs, core samples were taken and tested for concrete mix, unit weight, and compressive strength. The actual compressive strength was found to be 5,800 pounds per square inch (psi), the unit weight was found to be 115 pounds per cubic foot (pcf) and the mix was determined to be a sand-lightweight mix.

Ground penetrating radar was used to determine the number of, location of, and harping pattern of the strands in the 1977 DTs. It was found that the strands were harped at two locations over the entire span.

Based on the knowledge of local DT manufacturing history, it was determined that the 1977 DTs most likely used stress relieved strands rather than low-lax strands.

In order to accurately model the predicted deflections, actual concrete compressive strength data was used in the analysis.

1991 DTs

For the 1991 DTs, core samples were also taken and tested for concrete mix, unit weight, and compressive strength. Actual compressive strength was found to be 6,100 pounds per square inch (psi), unit weight was found to be 115 pounds per cubic foot (pcf) and the mix was determined to be a sand-lightweight mix.

Ground penetrating radar was used to determine the number of, location of, and harping pattern of the strands in the 1991 DTs. It was found that the strands were harped once at the mid-span. The 1991 DT's were manufactured using low-lax strands.

In order to accurately model the predicted deflections, actual concrete compressive strength data was used in the analysis.

ANALYSIS

A commercially available computer program was used to perform the theoretical analysis in order to determine the most likely scenario for the loss of camber shown in Figure 3. The program calculates losses of prestress using either "PCI Recommendations for P/S Losses" (PCI 1975), herein referred as to the "Recommendations Method" or the PCI Design Handbook "Simplified Method" (PCI 2004). When the Recommendations Method is used, the deflection calculations are based on an advanced step-by-step rational method which considers the time-dependent material properties. The time-dependent concrete properties of creep, shrinkage, and strand relaxation follow ACI 209 (ACI 1997). When the Recommendations Method is used, the computer program increases the loss term for shrinkage (stress loss in the prestressing strand due to shrinkage of the concrete (USH)) to account for the impact of high-shrinkage concrete (PCI 1975).

The theoretical analyses were divided into two groups for which various cases were analyzed for both the 1977 DTs and 1990 DTs. The analysis cases included the following:

- A design check with the design loads to determine theoretical initial camber, theoretical dead load deflections and theoretical deflection due to long-term creep.
- Long-term deflection analyses using a theoretical sustained load that would cause similar deflections as those shown in Figure 3.
- Evaluation of the theoretical initial amount of prestressing loss (i.e. strand slippage) that would cause similar deflections as those shown in Figure 3.
- In addition, the analyses were performed using both, the Recommendations Method and the Simplified Method.

From the theoretical analyses it was determined that the deflections found to be in excess of 1.6 inches, were not caused by long-term creep due to the roof sustained load (self-weight and superimposed dead loading). That is, through the analyses it was determined that a sustained load (in addition to the DT self-weight and the roof's superimposed dead load) of more than 17 psf would have had to be present for an extended period of time (years) in order to cause such deflections. This additional sustained load would be equivalent to approximately 3.5 inches of standing water on the entire DT for years. However, no evidence of such load was found as whatever water ponded on the DT's typically evaporates in days in the dry Denver climate. Thus, it was concluded that the large deflections were not due to long-term creep induced by standing water.

With regard to the loss of prestess, the results showed that the amount of theoretical prestress loss that would cause the permanent deflections, shown in Figure 3, would have been approximately 38 percent. This amount of prestress loss far exceeds any measured loss in the industry and was unlikely the cause of such deflections.

After determining that, neither long-term creep nor strand slippage could have caused the permanent deflections shown in Figure 3, progressive ponding analyses were performed. The progressive ponding analyses showed that the probable load due to the rain event caused the DTs to deflect in excess of 6 inches. This deflection then lead to more water on the already deflected DTs and likely caused the strain in the prestress strand to exceed 1 percent, leading to a loss of prestress and consequently the permanent deflections shown in Figure 3. Thus, it was concluded the most likely cause of the excessive permanent deflection was an overload on the DT's due to both the magnitude of the rain event and the poor drainage conditions on the roof.

REPAIR

DT REPAIR

The authors considered many repair concepts one of which was an external post-tensioning system for the DTs that had excessive deflections – such repairs have successfully been implemented in the past. External post tensioning still required removal of building contents below damaged DTs, roofing system repairs, removal and re-installation of fire suppression systems, and removal and re-installation of certain mechanical and electrical systems. There were other concerns with external post tensioning, such as reduced clearance to the bottom of the DTs and loss of use time, that were critical to the use of the facility. Conceptual drawings were submitted to a repair contractor who ultimately determined that the costs associated with external post tensioning met, or exceeded, the costs associated with removing and replacing the damaged DTs; thus, new DTs were installed.

ROOFING SYSTEM REPAIR

The roofing system atop the damaged DTs required repair in order to obtain adequate drainage and prevent future ponding. It proved to be a challenge to properly drain the roof due to the difference in camber between the new DTs and the existing DTs. A new roof drainage system was designed and built which utilized several interior primary and secondary roof drains with new low points that were north of the historic south drains. By using available height at the south parapet wall, it became possible to achieve a minimum ¼ inch per foot slope on the roof above and adjacent to all the new DTs. Due to multiple peaks and valleys, the roofing system design proved to be difficult to build. Through attention to detail and the use of tapered substrate, the design was successfully implemented. To-date the new design has performed well.

LESSONS LEARNED

This case study highlights several lessons for building owners, architects, maintenance engineers, and structural engineers that inspect existing roof systems that are built with precast prestressed DTs. Obviously it is important to maintain roof drainage systems, but a lesson that may not be so obvious is to pay special attention to low areas that may exist in a roofing system due to loss of camber in the DTs. Whatever the cause of the loss of camber, there exists the possibility for progressive ponding on older warehouse-type buildings which often have poor slope and poor roof drainage systems.

A tour through aerial images which look down on the industrial areas of many cities in the United States will reveal multiple roofs with localized low areas as evidenced by apparent staining. Before design professionals, owners, or contractors begin work on such buildings the authors encourage them to think about life safety and the possibility of progressive ponding. In this case it is unfortunate that those involved in the reroofing process (which was reported to have been permitted) did not recognize the hazard.

Building owners, design professionals, contractors, and building managers should think carefully about re-roofing activities. During re-roofing activities overflow scuppers could be inadvertently covered. Re-roofing offers an opportunity to assess the health of the roof structure and consider adding more drains and overflow drains. When dealing with large expansive roofs with inadequate slope, consider adding new drains in locations where low areas may exist to mitigate the possibility of progressive ponding.

For the engineer or architect who has occasion to inspect and design retrofits in older buildings with precast prestressed DT roof systems, it is important to consider that older DTs may have lost a significant amount of initial camber and may not behave as expected. Designers should take the time to measure the actual camber that remains in the DTs and consider the implications of excessive prestress losses.

CONLCLUSIONS

Although prestress/precast concrete structures are generally resilient, older precast prestressed DT roof structures with poor (less than code minimum) roof drainage systems may be at risk for collapse under rain loads. Through analysis and investigation, the authors determined the subject DT roof collapse was due to a one time overload caused by a significant rainfall that occurred in a short time, coupled with an inadequate roof drainage system and poor maintenance of roof drains. It is fortunate that this collapse did not occur a few hours earlier or it likely could have resulted in a loss of life. The authors encourage design professionals, facility managers, owners, and contractors who work with older buildings to investigate roof drainage conditions and the possibility of progressive ponding to avoid possible property damage and/or loss of life.

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