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FROM MY EXPERIENCE IN USA. THE FORMER TERMINAL A IN AIRPORT "LOGAN"

Information presented in this article is about former Terminal A building reinforced concrete structures performance, load bearing capacity study of structures and the demolition project. That was one of the first buildings in USA were the most of flexural elements were cast-in-place post-tensioned reinforced concrete. The building Terminal A was constructed in 1968, became obsolete, did not satisfy modern technological requirements and was demolished in 2002.

Keywords: *post-tensioned reinforced concrete, load bearing capacity, monitoring, and demolition*.

Airport "Logan" is sadly known all over the world as the airport where flights initiated and resulted in terrible events of September 11, 2001. However, for me this airport is the place from which I was flying for business trips and vacations, as well as a place of many projects in which I was involved. I was involved in study and design of several terminals, of people mover bridges, performed construction services.

Some tasks presented in this article were performed to support temporary service of the old terminal building. The building was in service for 30 years, obsolete but had to be used until new building was designed and its construction began.

Fig.1. Terminal A Former Building

Building Terminal A (Fig. 1) was designed by Architect- MINORU YAMASAKI & ASSOCIATES, Structural Engineer- SEPP FIRNKAS ENGINEERING and constructed in 1968. It was one of the first buildings in USA were main structures were post-tensioned cast-in-place reinforced concrete.

Presented is a "detective" story where "main hero", Terminal A, was "killed" – demolished (Fig. 11) at the start. However, the story is: what had happened before this.

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[1]*В відомостях про автора в науково-технічному збірнику "КОМУНАЛЬНЕ ГОСПОДАРСТВО МІСТ 124/2015" допущена неточність: Марк Янкелевича не було вписано до списку вчених США, у 2002/2003 роках, а було внесено в "America's Registry of Outstanding Professionals" - книгу, що містить відомості про професіоналів за різними напрямками.*

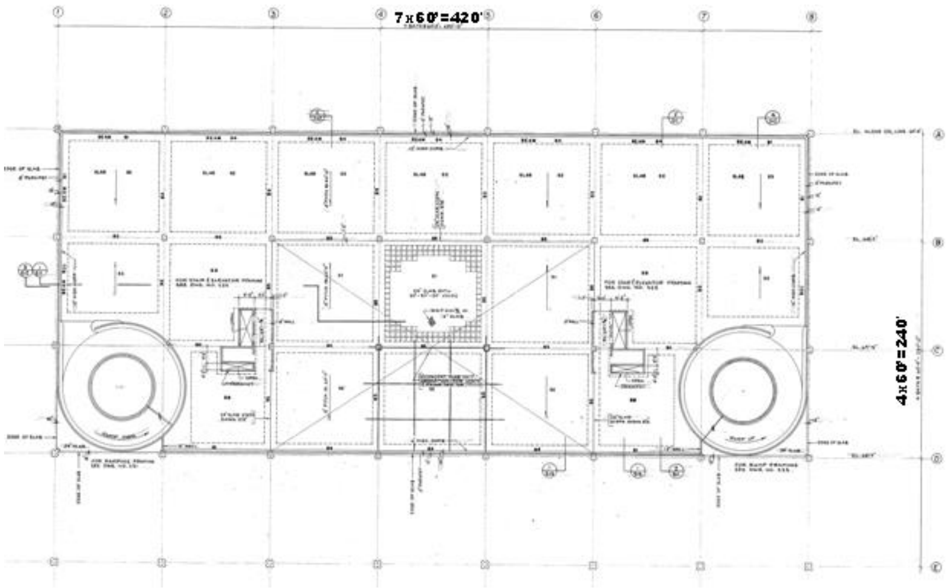

Fig.2. Typical Garage Floor of Terminal A

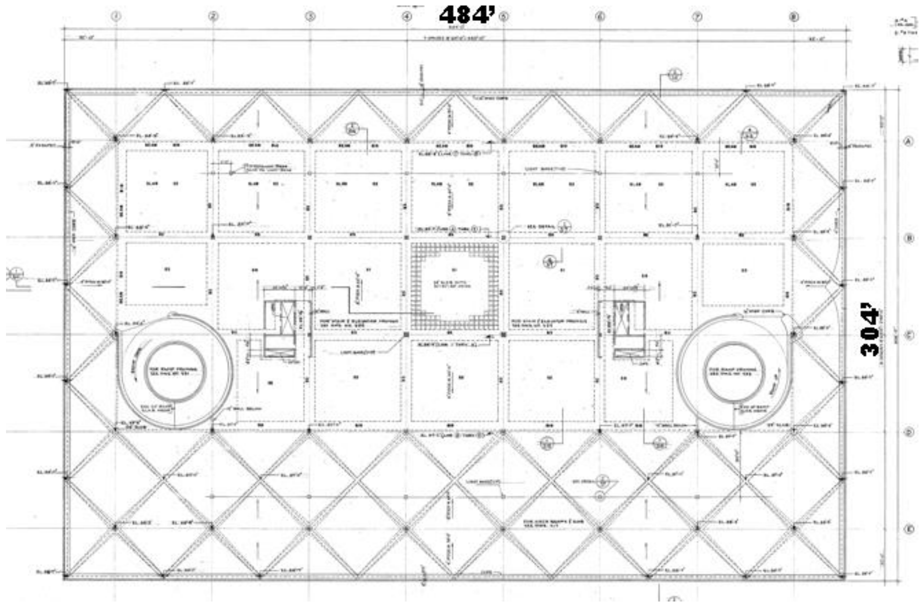

Fig.3. Roof Plan of Terminal A

The building presented (Fig. 2) was supported by columns with grid 60 x 60 feet (≈ 18 x 18 m.). As in typical terminals, there were the Departure and Arrival floors (2 lower floors), and also it had four parking garage floors and roof. The roof had additional parking areas at the total perimeter: the cantilevers on the west, south and east sides and an additional span with a total width canopy supported by the row of high columns on the north side (Fig. 3, 4).

The first task assigned to Weidlinger Associates in 1997 was the Wind Vulnerability Study. We were informed that the new project will be developed for Terminal A. However, the existing building should be in service about 4 years until it would be demolished and the new building would began to be constructed. The project of the existing building was developed in time when the wind and live loads used in design were smaller than should been used per Codes [1] of present time. The assignment was to verify that the existing building could sustain the required per current Codes design wind and live loads for the future 4 years.

The first view on the plan of building (Fig. 2) showed that monolithic ramps placed between two concrete circular walls were located at the each side of the building. These walls were interconnected at 5 levels with monolithic floor slabs and roof. Since this at first the wind load seemed to be not a problem. However, a further study showed that one of the outer circular walls of rump were not supported by the foundation but placed on columns at the first level.

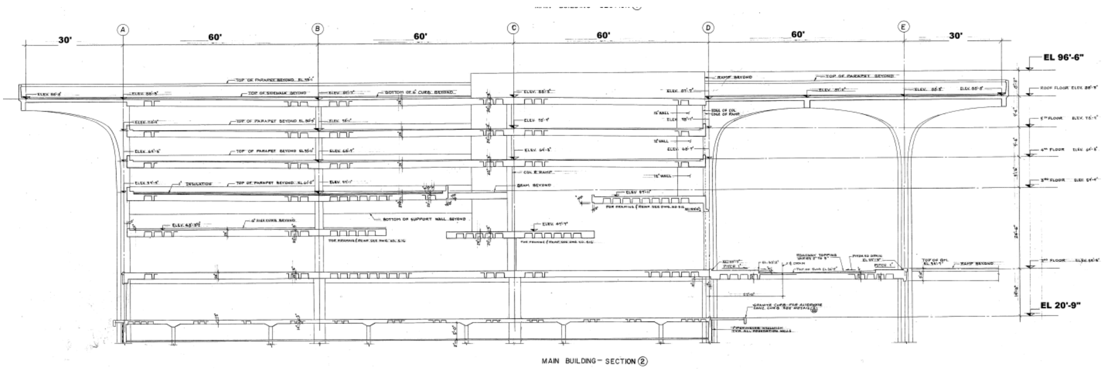

Fig.4. Cross Section of Building

Taking this into account it was decided to perform analysis of the regular frame supporting dead, live and wind loads at the 60 feet tributary area width.

The elastic frame with concrete columns and 6 level girds that were presented by solid portions of waffle slabs (Fig. 5) located at each of the columns rows were modeled and calculation was performed using STAAD software.

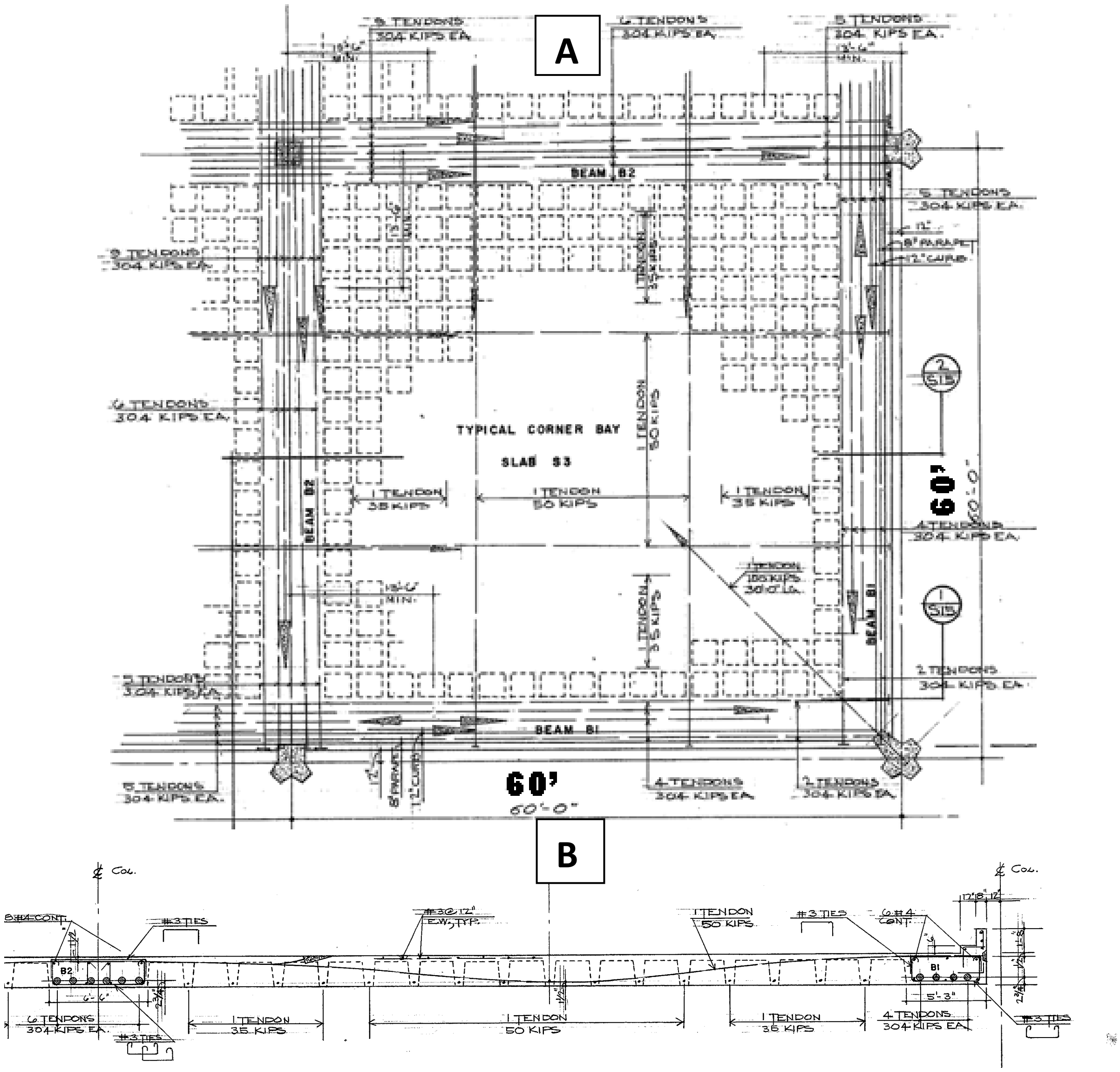

Fig.5. Plan (A) and Section (B) of a Corner Waffle Slab

Checking the concrete sections under the factored designed forces we revealed that the outer columns connections to the girds at the first three levels had the moments that were up to 15% larger than the capacity of sections while the forces in the joints at middle columns and girds sections were more than 50% smaller than the capacity of their sections. Inspection of the structures was performed and revealed that the edge columns had visual cracks. No cracks were found in interior sections. It was decided to place at the design model plastic hinges in connection of girds to the outer columns and applies in hinges the moments corresponding to the minimum moment capacity of columns or girds. Reviewing the forces in all critical sections obtained by this analysis under all design load combinations we found out that the load capacity of frame was in the allowable limits. Based on this and taking into account the 30 years of framing service we made a positive conclusion on the results of Wind Vulnerability Study.

While performing this study one day in 1998 we received a call from the terminal service personal that something happened at night in one of the building rooms. Inspection of that room showed that on the second floor in cafeteria kitchen the floor tiles were popping out. The visual inspection of the waffle slab soffit at this place had not shown any damage or even cracks in the ribs of the slab. It was concluded that this was not a structural problem. It was assumed that the often watering of this floor could some way expand the mortar under the tiles that caused their popping. The floor was repaired.

However, after the year and a half passed, the similar accident happened at the day time. The staff heard a sudden sharp noise and felt vibration after which the tiles popped out at the same place. It was realized that something happened with the waffle slab reinforcement.

The 4” thick waffle slab reinforced with #3 (≈10 mm diameter) at 12” (≈305 mm) had ribs spaced 3x3 feet (≈914x 914 mm) of total depth 2 feet (≈610 mm) and average width 7” (≈178mm). The ribs were reinforced with the post-tensioned 7 wire tendons 0.6” diameter (≈15mm) which were greased and did not have cohesion with concrete – were un-bonded.

It was a possibility that at least one of 5 mm wires in a tendon was broken creating a noise and vibration. It was a mechanical room under the kitchen room. The supporting post was promptly installed directly under the place of tiles popping.

It was made a decision to analyze the waffle slab. First the slab with local supporting post was checked. The post was installed near the span diagonal, about 13 feet (≈ 4m.) from the center of interior column.

The maximum forces at slab sections based on the elastic slab analysis were not larger than load bearing capacity in the critical sections.

However, the most critical was the corner slab that was not continuous on two corner exterior sides of the building. On one of the upper floors the popped up tiles were also found out in the closed at the most of time storage room located at the corner slab (Fig. 6).

The elastic analysis of waffle slab was performed using the model that included four slab units on 9 columns below and above in which interior sides were moment restrained (continuous) and exterior sides between the columns were free. The results of analysis showed that the span positive moment in the critical span section of the corner slab was about 30% larger than the moment capacity of the critical section.

Based on such results, we performed the yield line analysis of this slab calculating the moment capacity of sections using f_{ps} - stress in prestressed tendons at nominal strength [2] and project specified stresses of concrete. The load bearing capacity of the slab based on this analysis was equal of: P_{limit} = 0.365 ksf. The service load on the airport slab included 0.138 ksf dead load and 0.1 ksf live load – 0.238 ksf total load. The safety factor obtained was equal of SF = 1.53, which was 8% smaller than minimal safety factor that would be provided by the load factor design:

$$SF = (1.4DL + 1.6\ LL)/[\varphi(DL+LL)] = (0.138 \times 1{,}4 + 0.1 \times 1.6)/(0.9 \times 0.238) = 1.65.$$

Taking into account that yield-line analysis does not comply with the standard design practice in the USA and even this analysis shows about 8% overstress under the design load, it was decided to perform additional study of slabs behavior:

- The crack width gages were installed on cracks at several slab ribs (Fig. 7) and the crack width monthly monitoring were performed during 6 month period.
- Monthly survey was performed on random bays of floors to monitor the deflections variation during 6 month period too.
- Observations were made to figure out the real maximum live loads on the floors.

Fig.6. Popped Tiles on Corner Slab

A research laboratory was hired to perform first two tasks. The load observation was performed by counting the equipment weights and visiting the terminal areas at most critical days on the eve of government holidays. The maximum load that was estimated at the crowded departure floor occurred to be not more than 30 psf (pounds per square foot) ≈ 150 kg/m^2, while the design live load at airport by Code used in calculations was 100 psf (≈500 kg/m^2).

Since the cracks width and the deflections during the half year observation had not been increased and the observed load was much less than the design load, we concluded that the airport building could stay in temporary service.

However, some other problems emerged during the observation of building structures.

The two spiral ramps for car traffic to parking garages located at the upper floors at the west and east edges of the building were designed different way (probably for research goals). The east ramp slab had non-prestressed reinforcement mesh while the west ramp slab was reinforced with radial prestressed tendons.

Fig.7. Monitor on the Crack on the Waffle Slab Rib

The observations of ramps showed that in several places the both ramp slabs had deterioration of concrete with open reinforcement covered with rust (Fig. 8). It should be noted that these ramp slabs are actually 8” (≈ 200mm.) thick one way slabs with 16 ft (≈ 5m.) span restrained on both sides in circular 1 foot (≈ 30mm.) thick walls and in such arrangement restricted not only from rotation but also from horizontal movement.

The investigations of such slabs [3, 4, 5] show that their capacity drastically increases due to outward trust. Our calculations, performed using the algorithm that was developed in NIISK (Kiev) for program “RASPOR” [5], showed that the use of prestressed reinforcement in such slabs was too redundant and the required capacity was achieved even if the amount of reinforcement was 75% lesser than what was used in the original design.

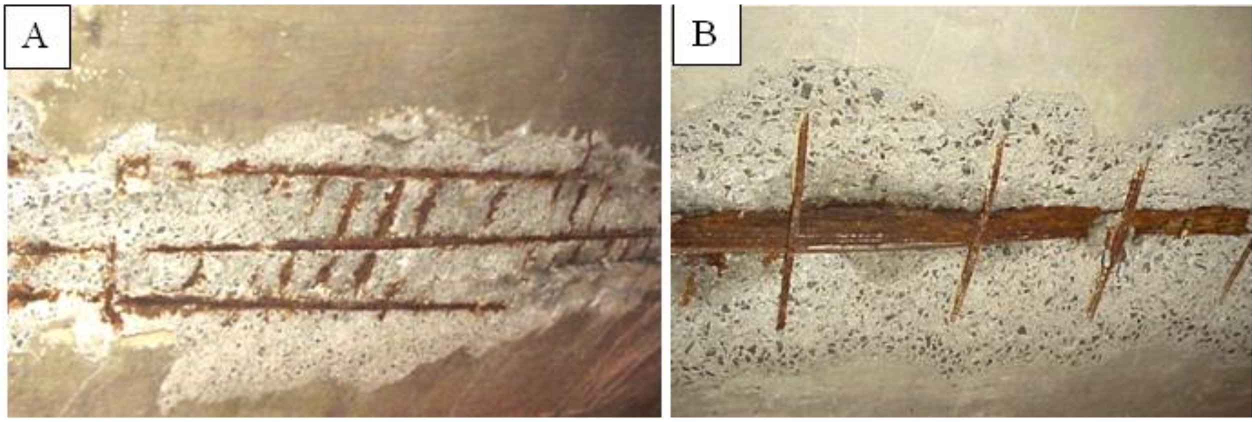

Fig. 8. Deteriorated Concrete and Rusted Reinforcement at the Bottom of Ramp Slabs:
A – East Ramp; B –West Ramp

Based on above, it was recommended to use rust remover and to clean rust at the exposed reinforcement and apply protecting paint. After this temporary use of ramps for the term required was allowed.

The next task was verification of a partial demolition option at the north-east corner of cantilevered roof. This region was considered for the early beginning of the new Terminal A construction without termination of old Terminal A service. For design such temporary demolition procedure without destruction of other roof spans it was required to verify that the existing post-tensioned reinforcement and its anchors were in good condition. High pressure hydraulic demolition procedure was used for concrete chipping and exposing the tendons and anchors. As it is shown on Fig. 9 the anchors and the slab reinforcing at the roof corner were in good condition. There was an option to re-anchor these tendons that should stay in place before start of slab partial demolition.

After the demolition at cantilever corner the middle-span moment at the next span of the roof rib supported by the corner column would increase. The decision was to add the steel beam above and connect it with hangers to the rib, as it is shown on the design model (Fig. 10). Such way the capacity of the partially demolished roof was warranted.

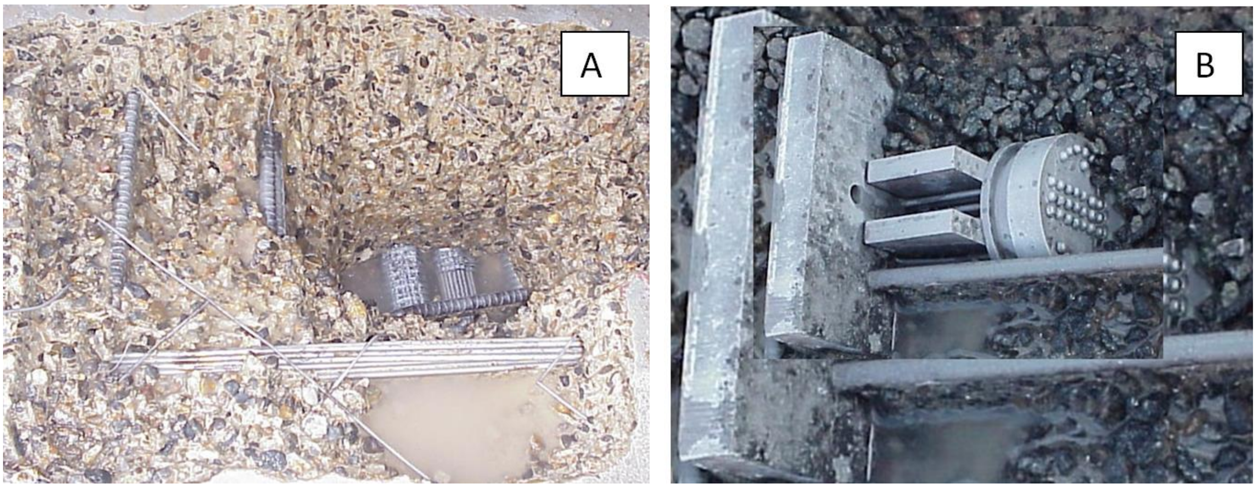

Fig. 9. Open Reinforcing (A) and Anchor (B) after Concrete Chipping

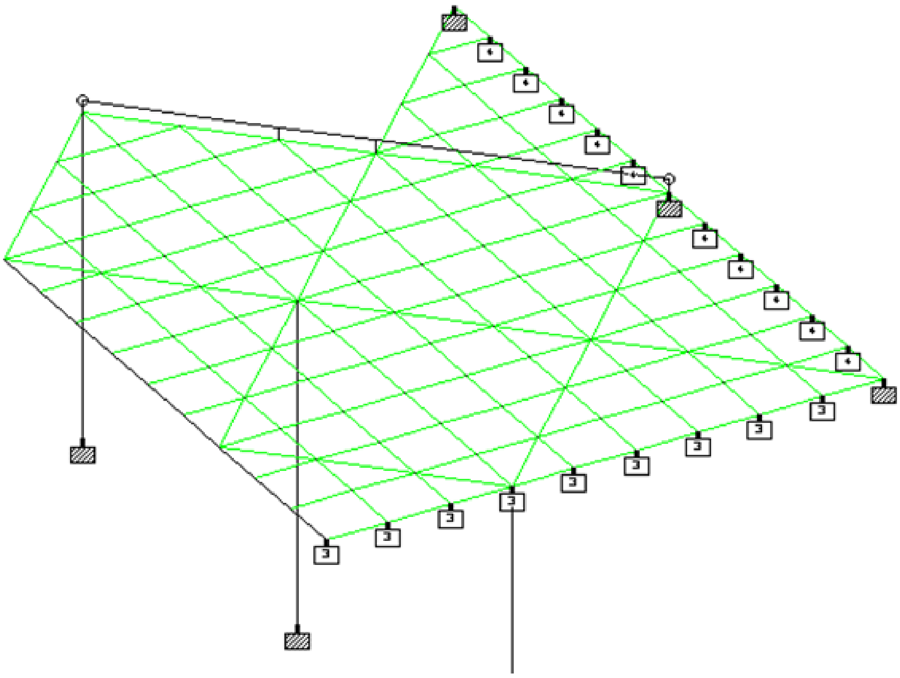

Fig.10. Model of Roof Corner after Cantilever Demolition for Analysis with Program STAAD.

The last task assigned to Weidlinger Associates was a design procedure for building demolition. The main goal of this design was to avoid progressive collapse of the building and to develop a demolition sequence preventing any dust and debris from getting to take-off and lending runways of the airport in service.

A detailed step by step demolition sequence was developed in such way that at each step the assigned by design portion of structure should be brought down. To achieve such goal for totally cast in place prestressed concrete structure was a very complex task. The demolition project was developed and coordinated with an experienced demolition company who performed the demolition. One of my tasks was to visit the demolition site from time to time and to control the demolition process. The photos that I made during my visits are presented on Fig. 11. The demolition mostly performed by using crane boom swing with a heavy weight hanging ball.

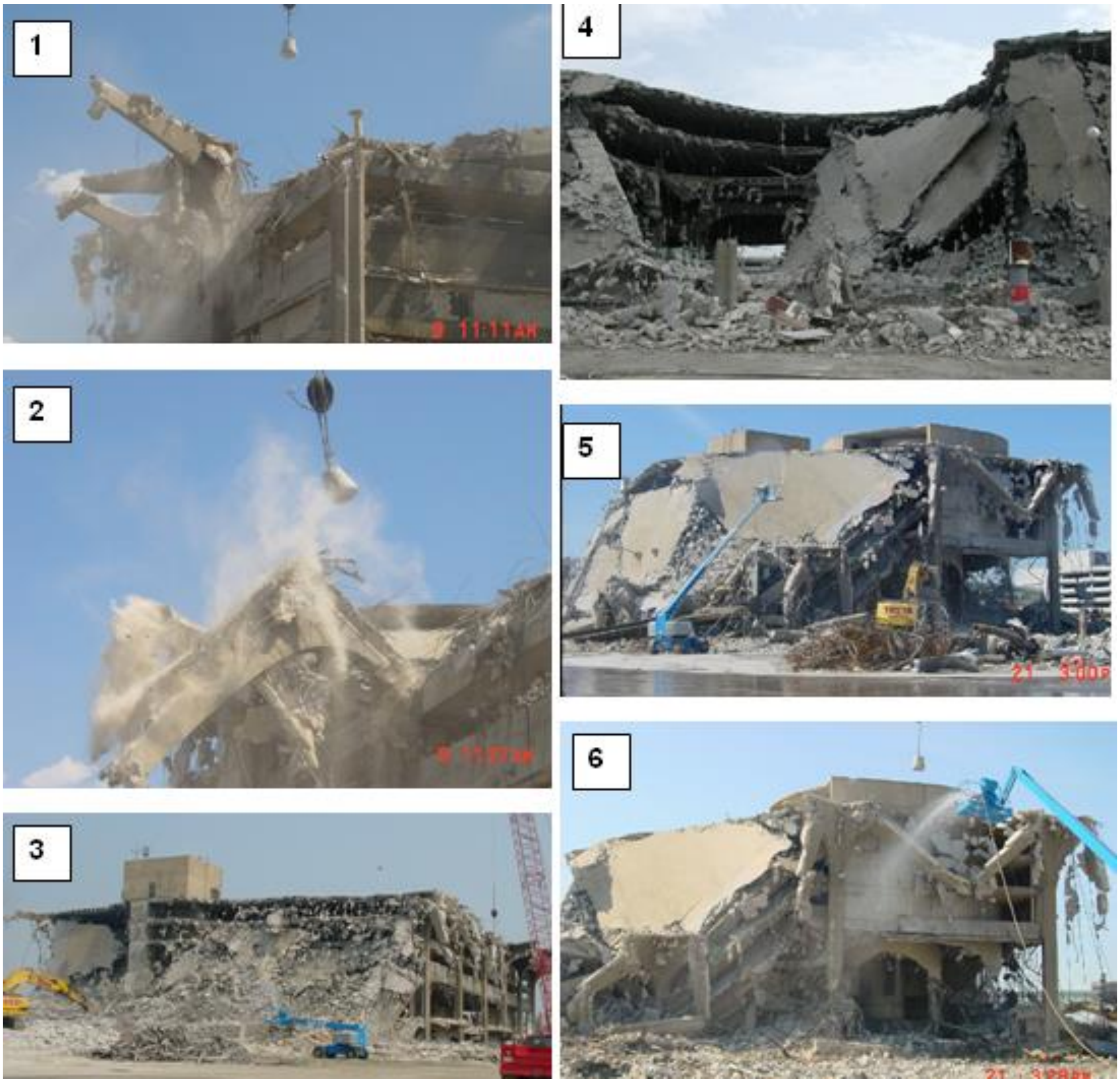

Fig. 11. Phases of Demolition (sequence follows the numbers)

The procedure took place with permanent water streaming around the each particular demolition place to avoid dust and small debris to fly around the airport area. The demolition was performed approximately during a month period and finally was completed in August 2002.

Soon after this the construction of new Terminal A started. The new terminal project design was completed before the old terminal demolition. The structural design of the new terminal was also performed by our company and I took part in the design.

The new terminal was opened in March 2005 and that year I flied from this terminal for the business trip to Atlanta.

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ИЗ МОЕГО ОПЫТА В США. БЫВШЕЕ ЗДАНИЕ ТЕРМИНАЛА А АЭРОПОРТА «ЛОГАН»

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Сведения о натурном обследовании, об исследовании несущей способности пост-напряженных железобетонных конструкций Терминала А Бостонского аэропорта "Логан", а также о проекте разборки здания представлены в этой статье. Терминал А являлся одним из первых построенных в США зданий, основные изгибаемые конструкции которого выполнены из монолитного железобетона с натяжением арматуры на бетон. Здание Терминала А было построено в 1968 году и разобрано в 2002 году в связи с тем, что оно больше не соответствовало современным технологическим требованиям.

Ключевые слова: *пост напряженный железобетон, несущая способность, натурное наблюдение, разборка.*

З МОГО ДОСВІДУ В США. КОЛИШНЯ БУДІВЛЯ ТЕРМІНАЛУ А АЕРОПОРТУ «ЛОГАН»

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Відомості про натурне обстеження, дослідження несучої здатності пост-напружених залізобетонних конструкцій Терміналу А Бостонського Аеропорту "Логан", а також про проект розбирання будівлі представлені в цій статті. Термінал А був однією з перших побудованих в США будівель, основні згинаючі конструкції якої виконані з монолітного залізобетону з натягом арматури на бетон. Будівлю Терміналу А було побудовано в 1968 році і розібрано в 2002 році в зв'язку з тим, що вона більше не відповідала сучасним технологічним вимогам.

Ключові слова: *пост напружений залізобетон, несуча здатність, натурне спостереження, розбирання.*