

Research Progress and Accomplishments

2000 – 2001

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Papers
Chronicling
Technical
Achievements
of the
Multidisciplinary
Center for
Earthquake
Engineering
Research*

Research Progress and Accomplishments

2000 - 2001

Multidisciplinary Center for Earthquake Engineering Research
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Seismic Evaluation and Retrofit of the Ataturk International Airport Terminal Building

by Michael C. Constantinou, Andrew S. Whittaker and Emmanuel Velivasakis

Research Objectives

MCEER was part of a team charged with the seismic upgrade of Istanbul's Ataturk International Airport Terminal Building. The project involved analysis of post-earthquake damage to the Terminal Building following the 1999 Marmara, Turkey earthquake, and development of a retrofit scheme. Nonlinear static and dynamic analyses were conducted using procedures set forth in FEMA 273 (including numerous contributions from MCEER researchers). Pushover analyses were conducted using IDARC (developed with MCEER support). Dynamic analyses of the roof-isolated structure considering inelastic frame behavior were conducted using computer software based on modifications of program 3D-BASIS (developed with MCEER support). Several retrofit schemes were investigated, including conventional methods and materials, and new technologies. The scheme selected included isolation of the roof trusses, addition of shock transmission units, and selected retrofit and strengthening of reinforced concrete construction.

The Turkish build-operate-transfer consortium, TEPE-AKFEN-VIE (TAV) and its advisor, New York-based Turner International, approved the evaluation and retrofit scheme. Members of the team were LZA Technology, a division of Thornton-Tomasetti Group (leader), Michael C. Constantinou and Andrew S. Whittaker (formerly of PEER), both of the University at Buffalo, and Tuncel Engineering and Fondsiyuon Muhendislik Insaatvetic Ltd., both of Istanbul. The project is an excellent example of bringing research results to implementation.

At the time of the August 19, 1999, Izmit earthquake, the new Ataturk International Airport Terminal building was nearing completion. The airport, which is located 25 km from the center of Istanbul, was shaken and damaged by the earthquake. (The airport is located approximately 70 km from the fault rupture plane.) The new Terminal building is a three-story reinforced concrete building with a space-frame roof. The plan footprint is approximately 240 m by 168 m. A view of the terminal building is presented in Figure 1.

Sponsors

LZA Technology, Thornton-Tomasetti Group

National Science Foundation, Earthquake Engineering Research Centers Program

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- **Sani Sener**, CEO, and **Gokhan Ozber**, Deputy General Manager, Tepe Akfen Vie (TAV), Investment Construction and Operation Inc., Turkey (Airport Owner)



■ **Figure 1.** New Ataturk International Airport Terminal Building

The lowest story of the building provided mechanical and baggage handling services. The second and third stories of the building housed the Arrivals and Departures Halls, respectively. A plan view of the building is shown in Figure 2a. As shown in Figure 2a, the 240 m by 168 m building is composed of 20 pods (or independent frames); the typical pod dimension is 48 m by 48 m. A typical pod is shown shaded in this figure. The pods were separated by 50 mm wide expansion joints (EJs). Figure 2b shows the framing in the third and second stories of a typical pod; the framing in the first story was similar to that in the second story. Figure 2c is a cross-section through the build-

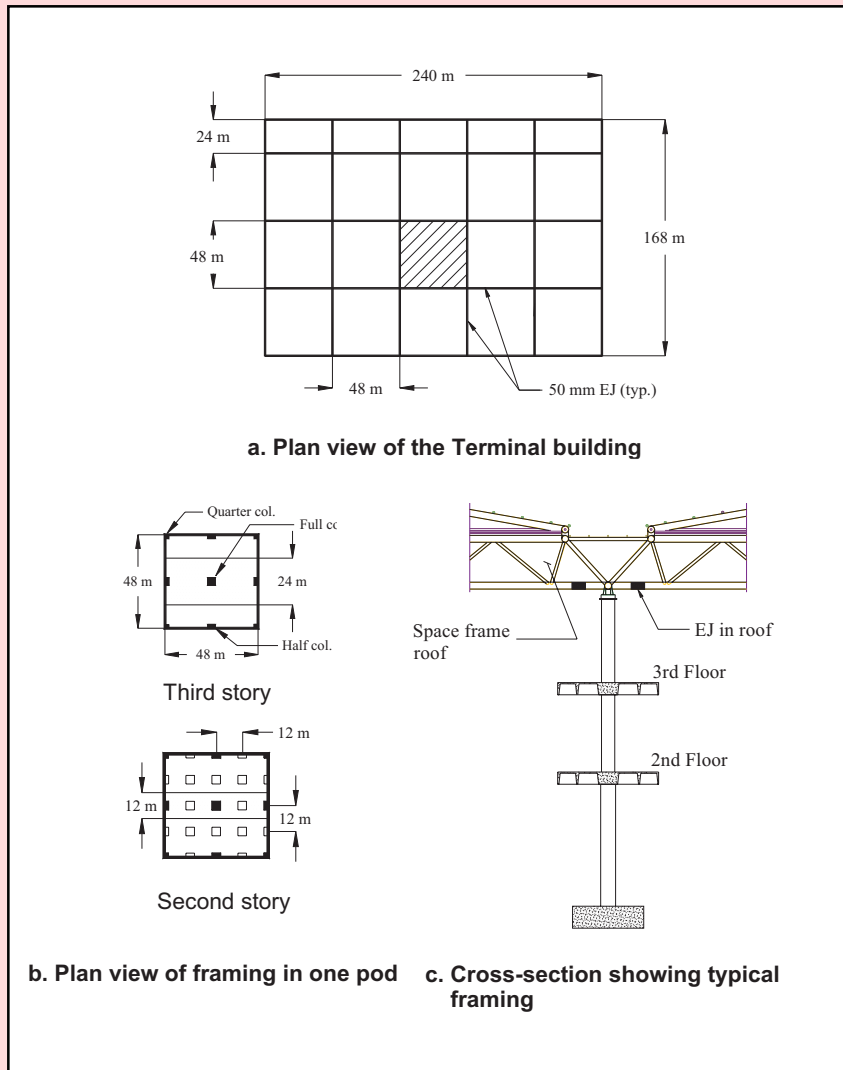
ing showing typical framing. The building is framed in reinforced concrete. Above the third floor level, cantilever columns on 24 m centers supported a three-dimensional steel space-frame roof structure. The space-frame roof was equipped with sleeved movement joints to permit thermal expansion and contrac-

tion of the roof. These movement joints did not align with the expansion joints in the reinforced concrete framing. At the third floor and below, gravity loads were supported by reinforced concrete waffle slabs and columns at 12 m on center. Lateral loads were resisted by waffle-slab moment-frame construction in each direction. Solid beams of a depth equal to that of the waffle slab spanned between the columns. Around the perimeter of each pod, the column sizes were substantially reduced from those in the interior of the pod. At the corners of each pod, the four columns were approximately square but with dimensions one-half of those of the interior columns (and called

It is anticipated that this project will be of interest to building owners in seismic regions throughout the world. Three different retrofit approaches were taken, to strengthen various sections of the building. The performance of the seismic isolation and lock-up devices in future seismic events will provide additional data on their reliability and effectiveness for earthquake hazard mitigation. The entire evaluation and subsequent installation of the retrofit scheme was accomplished in less than four months.

Links to Current Research

- The seismic evaluation procedures used in this project were developed with full or partial MCEER support, including IDARC 4.0, 3D-BASIS, FEMA 273/274 and NEHRP 2000.



■ Figure 2. Construction of the Terminal Building

quarter columns in this paper). Along each edge of each pod, and between the corner quarter columns, the columns were approximately rectangular and half the area of the typical interior columns (and called half columns in this paper.)

During the August 19, 1999 earthquake, the Terminal building was subjected to modest earthquake shaking. The maximum recorded horizontal ground acceleration recorded at the Airport was approximately 0.1 g; the maximum vertical

acceleration was less than 0.05g. Investigations by LZA engineers immediately following the earthquake identified damage to parts of the building, including spalling of the cover concrete and buckling of the longitudinal rebar at the base of the third story columns (showing also lap splices and little transverse rebar in the hinge zones), loss of concrete at the underside of the roof truss-cantilever column connections and slippage of the roof-truss baseplates atop the columns,

Web Sites

Overall Project Information:
IZA Technology:
<http://www.lzagroup.com>

Hardware Information:
Earthquake Protection Systems:
<http://earthquakeprotection.com>
Taylor Devices:
<http://www.taylordevices.com>



■ **Figure 3.** Observed Damage to the Terminal Building; base of third story column (left) and base of roof truss connection to column (right)

and splitting cracks and spalled concrete in the beam-column joints at the third floor level (indicating no transverse reinforcement in the joints). Photographs of damage to the building are shown in Figure 3.

Seismic Evaluation of Terminal Building

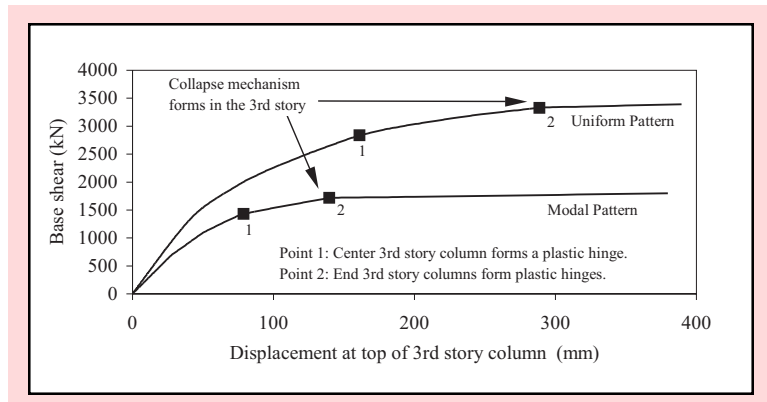
One frame in one of the typical 48 m by 48 m pods was selected for evaluation by nonlinear static analysis. The objectives of the analysis of the existing building were to (a) correlate the locations of the observed damage and that predicted by analysis, (b) to estimate the displacement capacity of the existing framing system, and (c) to provide guidance to the design team on plausible retrofit schemes. The lightly shaded zones in Figure 2b indicate the location and width of the sample frame. The central bay of framing was selected because it included the third story cantilever columns that were damaged during the earthquake. This frame was 24

m wide above the 3rd floor level and 12 m wide at that level and below.

The nonlinear static (or push-over) analysis was conducted using the procedures set forth in FEMA 273 (FEMA 1997). The existing framing was modeled using the as-built construction drawings. The tributary widths of the waffle-slab or beam framing for stiffness calculations was set equal to 12 m; the strength of the beam framing was based on the reinforcement in the solid segments of the waffle slabs between the column. The third-story columns were linked at the roof level by rigid axial elements to simulate the effect of the space-frame roof structure. The deformation capacities of the reinforced concrete components were set equal to the values listed in Chapter 6 of FEMA 273 for reinforced concrete columns, beams, and beam-column joints. Because little transverse reinforcement was provided in the critical or hinging regions, the deformation capacities of all components were established

assuming non-conforming transverse reinforcement. A sample calculation for the third-story column showed a maximum plastic rotation of 0.01 radian for the performance level of collapse prevention because the axial load ratio was less than 0.1, the transverse reinforcement was non-conforming, and the shear force ratio was less than 3. A target displacement for the pushover analysis was established based on revised criteria established by the owner after the August 19 earthquake, namely, a spectrum with ordinates equal to 150 percent of the elastic spectrum set forth in the 1997 Turkish seismic code for the site of the airport. The resulting target displacement at the roof level was 230 mm for an elastic period of 1.25 seconds.

The pushover analysis was accomplished using IDARC 4.0 (Valles et al., 1996). The existing building was analyzed using two lateral-load profiles; second-order effects were automatically included. For this frame, both a uniform pattern and a modal pattern were used. The collapse mechanism involved hinging of the third story cantilever columns for both loading profiles. Figure 4 presents the base shear-roof displacement relationships for the two lateral-load profiles. The significant difference between the two curves is due to the large differences between the weights at the roof and lower levels and the resulting differences between the loading profiles. Nearly all of the building deformation occurs in the third story of the building with the modal load pattern and the framing below the third story does not yield. Such a distribution of predicted damage is completely consistent with the observed damage



■ Figure 4. Pushover Analysis of a Frame in the Original Building

to the reinforced framing with one exception: the beam-column joint damage was not predicted because these joints were assumed to be rigid for the analysis. For information, the third-story drift corresponding to a plastic rotation of 0.01 radian in the third-story column was 80 mm. Clearly, the deformation capacity of the third-story columns would be exhausted well before the target displacement at the roof level was achieved. Further evaluation of the existing building showed that the moment-resisting frames were undesirable weak column-strong beam frames.

Conventional and Protective Systems Retrofit Concepts

Retrofit schemes for the Terminal building were developed using conventional methods and materials and new technologies. The conventional retrofit options considered by the design team all made use of a new ductile lateral-force-resisting system, including steel braced frames, special reinforced concrete shear walls, and special reinforced concrete moment frames. All of the conventional retrofit schemes in-

cluded new foundations under the new lateral-force-resisting components, repair and reconstruction of the third-story column to roof-truss framing, elimination of the expansion joints between the pods, joining and jacketing adjacent quarter and half columns, and jacketing of the interior third-story columns. The addition of a new lateral-force-resisting system was not considered feasible because substantial new vertical elements (walls or braced frames) could not be installed in or below the first story.

New technologies in the form of seismic isolation and supplemental damping were evaluated for the retrofit of the Terminal building. Because the addition of supplemental damping devices would have involved the addition of braces or wall panels in the lower two stories of the building, a detailed retrofit design using supplemental dampers was not prepared. Two seismic isolation options were considered: base isolation of the entire building, and isolation of the roof trusses. The building isolation option was the preferred option of the two, but was rejected by the owner because of the advanced state of construction of the building. The installation of an isolation system immediately above the foundation would have required the demolition and reconstruction of the ground floor of the Terminal building, and the removal and reinstallation of the mechanical and baggage handling systems located in the first story of the building. The second isolation option was studied in detail, and was selected for the retrofit of the building by the owner. Information on this retrofit scheme is presented below.

Retrofit (Upgrade) of the Terminal Building

This scheme selected for the retrofit (or upgrade) of the Terminal building involved the isolation of the roof trusses to reduce the demand on the third story columns and the framing at the lower levels, the addition of shock transmission units to the roof trusses to lock the space-frame truss pods together during earthquake shaking so that the space frame would act as a diaphragm, and selected retrofit and strengthening of the reinforced concrete construction as summarized below.

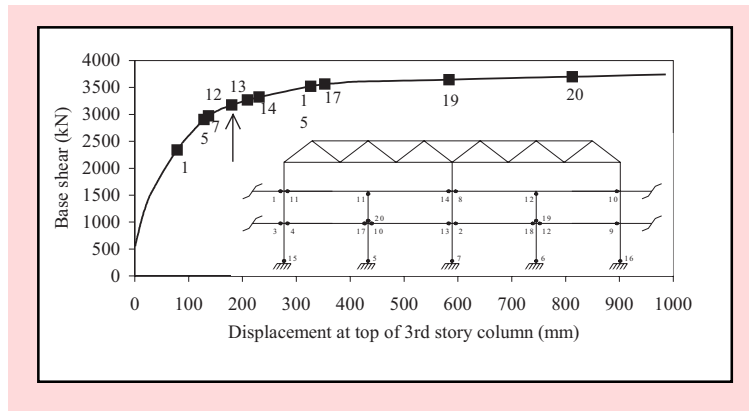
Because of architectural constraints, the size of the third story columns could not be increased substantially, so the existing flexural strength of these columns as cantilever elements dictated the inertial force that could be developed at the roof level. Fuses in the form of Friction Pendulum (FP) isolation bearings were used to limit the lateral forces that could be imposed on the third-story columns. (FP bearings were used because such bearings can isolate light components and structures.) Preliminary calculations called for an isolated period of 3.00 seconds (based on the radius of the sliding surface), a design friction coefficient of 0.09, and a displacement capacity of approximately 260 mm.

The quarter and half columns around the perimeter of each pod were joined to the adjacent quarter and half columns respectively using reinforced concrete. Additional vertical reinforcement was placed in the joints between the part columns and around the perimeter of the joined columns to

further strengthen the columns. The objective of such strengthening was to eliminate the weak column-strong beam framing. These columns and the modestly strengthened interior square columns were then jacketed with circular steel casings to substantially increase the shear strength of all the columns and to provide confinement in potential hinge regions. Such column strengthening was implemented in the second and third stories. In addition, the expansion joints between the pods at the second and third floor levels were eliminated by tying the pods together with reinforced concrete components to substantially increase the redundancy of the lateral-force-resisting system. No beams at either the second or third floors were strengthened.

The performance of the retrofitted building was checked by nonlinear static analysis of the frame described above. The resulting pushover curve for the modal load pattern is presented below in Figure 5 together with a sketch showing the sequence of plastic hinge formation (1 through 20). Note that some hinges form simultaneously. For a roof displacement less than 500 mm, hinges form in the beams, at the base of the columns above the foundation, and in the two-story columns at the underside of the third story only.

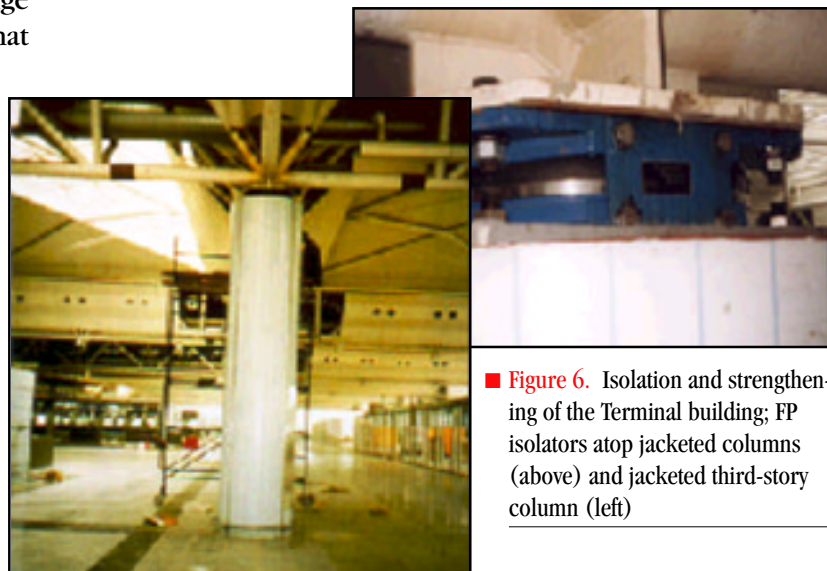
The retrofit design was further evaluated by nonlinear dynamic analysis using 20 ground motion records that matched on average the revised design spectrum described above. The mean maximum displacement at



■ Figure 5. Pushover Analysis of the Retrofitted Frame

the roof level of 190 mm is indicated by the open-ended arrow on the pushover curve of Figure 5. The deformation demands on the beams and columns in the retrofitted building frame at this roof displacement were considered acceptable for the performance level of collapse prevention.

Two photographs of the retrofit work showing an installed FP bearing (before release) and a jacketed column (covered by an architectural treatment) are presented in



■ Figure 6. Isolation and strengthening of the Terminal building; FP isolators atop jacketed columns (above) and jacketed third-story column (left)

Figure 6. No retrofit work was undertaken in either the first story or to the foundations. The nonlinear static analysis and the nonlinear dynamic analysis indicated no inelastic action in the first story above the footings, in part because the columns in this story were substantially stronger and taller than the columns in the stories above. Retrofit of the footings beneath the existing columns was not feasible given the advanced state of the construction at the time the retrofit scheme was developed.

Summary and Conclusions

An innovative retrofit and upgrade scheme was developed and

implemented for the new Ataturk International Airport Terminal building, which was damaged during the August 19, 1999, Izmit earthquake. The retrofit scheme involved the use of conventional strengthening and seismic isolation hardware to avoid building collapse in the event of a maximum earthquake. The efficacy of the retrofit scheme was demonstrated by nonlinear dynamic and static analysis. The studies of the existing building and the development of the retrofit schemes commenced in late September 1999, and the retrofit construction work was completed by the end of December 1999: a period of approximately 12 weeks.

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- Valles, R.E., Reinhorn, A. M., Kunnath, S. K., Li, C and Madan, A., (1996), *IDARC2D Version 4.0: A Computer Program for the Inelastic Damage Analysis of Buildings*, Report No. NCEER-96-0010, National Center for Earthquake Engineering Research, University at Buffalo, June.