WINGS OF ISOLATION

San Francisco International Airport's new terminal is protected by 267 steel seismic isolators

Shown above are two views of the new international terminal at San Francisco International Airport. Note how the terminal is built above an existing roadway.

Pictured on the opposite page (top) is a model shot of a full-size joint; shown opposite page (bottom) is a skylight under construction in the roof trusses.

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IT IS A VISION AND DREAM OF ARCHITECTURAL AND ENGINEERING DESIGN to intelligently design an aesthetic form symbolizing a signature structure of massive proportions.

Such a structure is the New International Terminal under construction at San Francisco International Airport. The glass and aluminum clad International Terminal is the $400 million centerpiece of the Airport's $2.4 billion expansion and modernization program. Its dramatic 860' long wing-like roof structure and 700' long and 80' tall glass wall can withstand a Richter Magnitude eight earthquake because of the 267 friction pendulum seismic isolators installed at the base of the building columns. The construction of the new terminal created a challenge because this signature building is located on a deep and soft site in one of the most severe earthquake zones in the country.

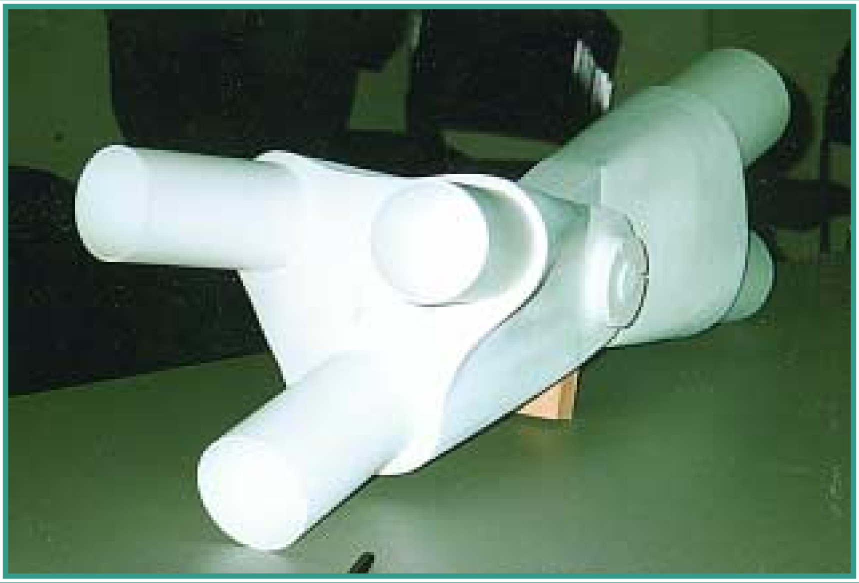
The problem was further compounded by the airport's directive to locate this new terminal over the existing entrance roadway to the domestic terminals without disrupting the traffic during construction. In addition, the design needed to minimize damage and disruption following a major earthquake. The structure of the new terminal is complete with all 267 bearings installed, and full operation is slated for mid-2000. With over 1.2 million sq. ft. of floor space and over 22 million cubic feet of interior volume, the New International Terminal is the largest base isolated building in the world, more than twice the size of the current largest base isolated building in Japan.

In early 1993, the San Francisco Airport Commission selected the Joint Venture Architects (JVA), comprised of Skidmore, Owings & Merrill, Del Campo & Maru, and Michael Willis and Associates from a design competition as designers for this signature building. Skidmore, Owings & Merrill (SOM) were also the structural engineers of the New International Terminal.

Shaky Ground

The elegant and yet delicate wing-like roof structure resting on 20 tall slender steel columns and the expanse of glass on the west wall of the departure hall presented a challenge for seismic design because of the existing site condition. The geological conditions at the expansion site are not ideal from an earthquake point of view. The site predominantly consists of bay mud under-lain by sand, silt, clay, and soft rock with depth of bedrock varying from 90' to 150' below the ground level.

The project geotechnical engineering consultants developed site specific seismic hazard evaluations for use in the analysis and design of the new terminal. These ground motions represented site specific response spectra curves for several levels of earthquake hazard were further revised and updated during the design process, incorporating the most current recommendations regarding recent earthquakes, effects of deep bay mud and proximity to nearby faults. Three earthquake levels (A, B & D) peak ground accelerations (0.34g, 0.6g, & 0.6g) were considered with probabilities of exceedance 50%, 10% & 10% in 50, 50 & 100 years, respectively. These earthquakes essentially represented a minor, major, and extreme earthquake event. Earthquakes B & D correspond to California Building Code design basis and upper bound earthquakes, respectively.



Pictured above and on the opposite page are the roof trusses and T-Y-K joints during fabrication and erection.

Pictured below is the cruciform column during fabrication.

Wing Roof Structure

The overall length of the main roof truss span is 840' with a center span column to column of 380'. The center truss spans 180' hinge to hinge. The design of the overall main roof truss and diaphragm structure resulted in an average of framed steel weight of 33 psf and 60 psf including roof construction. The critical nature of both the geometry of the main roof structure and use of the building required a higher structural design performance criterion.

In response, the design team developed the criteria that the main roof truss, diaphragm and supporting box columns remain elastic under the extreme earthquake ground motion of EQ-D (10% chance of exceedance in 100 years with a 1000 years return period). The design included both horizontal and vertical response spectra analysis resulting in design spectral shears of 78% horizontal and 100% vertical of main roof structure mass. Factors of safety greater than 3 were maintained under service loads and 1.5 under ultimate load conditions. Final member design was governed by T-K-Y joint design requirements for punching shear through branch members per AWS D1.1.

Proportioning of member design was balanced on both inside cantilever with main center span and outside cantilevered roof trusses. Maximum vertical deflections were 4.5" under dead plus live loads and 9" combined with peak seismic loading. Wind tunnel models evaluated at the Boundary Layer Wind Tunnel at the University of Western Ontario predicted peak 100-year wind pressures of less than 30 psf.

The main roof trusses consist of approximately 3,200 tons (6,400 kips) of steel pipe, plate, rods, and roof purlin tubing and include five sets each spanning a total of 250m (820') north to south. The westernmost truss is on the exterior just outside the "great hall" window wall. Each set consists of two double cantilevered one way trusses with a 24.4m (80') back span, an outside cantilever of 42.7m (140'), and an inside cantilever of 30.5m (100'). These are linked by a center span of 54.9m, (180') which has become known as the "football" truss. The links consist of a two piece cast steel pin-joint with 152mm (6") diameter pins. Truss centerline depths vary up to 8.8m (29') at the back spans and 7m (23') at the center trusses. Top and bottom truss chord sections vary from 305mm to 508mm (12" to 20") diameter and 21mm to 51mm (0.84" to 2.0") wall thickness. Above the spherical ball joint at the top of each of the cantilevered box columns which transfer both vertical and horizontal loads down to Level 3, truss chords and sway lateral bracing tie rods connect into the main 102mm (4") cruciform plated verticals. Diagonals and vertical pipe sections are spaced at 6.1m (20') centers with opposing diagonal tension rods of 76mm and 95mm (3" and 3.75") in diameter. The roof diaphragm consists of 115mm (4-1/2") acoustical metal deck in 6.1m (20') segmented spans with in-plane diagonal bracing and purlins.

Truss steel tubular T-Y-K joints featuring through thickness full penetration welds, with notch tough and low sulfur material properties - API 5L pipe and API 2H plate, were designed and fabricated per AWS D1.1. The roof structure was designed for both horizontal and vertical seismic response spectra to remain elastic. Skylights directly above the top chord extend the entire length as well as between the top chords of the center trusses. Artist James Carpenter has designed translucent tensile structures, which will hang below these openings to act as light reflectors. The main truss aesthetic, proportioning and profile, were continually refined throughout the design phases to achieve a maximum integration of form and economy

Roof Fabrication

The project's fabricators, AISC-member Herrick Corporation of Pleasanton, CA, and AISC-member PDM Strocal, Inc., in Stockton, CA, was able to achieve fabrication and erection tolerances for the main roof structure to within +/- 1" as specified on the design drawings. This included calculation of dead load deflections and detailing working points to achieve cambers resulting in final erected elevations within tolerance.

Herrick subcontractor NESCO/XKT Engineering, which had unique experience in tubular steel offshore platforms, fabricated roof trusses at a converted Naval Shipyard facility at Mare Island at the north end of San Francisco Bay. An automated computer controlled contour flame cutting apparatus was utilized to bevel and fit-up all T-Y-K pipe to pipe connections based on theoretical working points. Mill pipe out-of-roundness was corrected in the shop using large presses to reform pipe diameters at joint locations. Shop fabrication tolerances of +/- 3/8" were achieved on the highly geometrically irregular three-chord center "football" trusses.

Roof Erection

Roof trusses were fully assembled in the shop and then disassembled into some 35 major pieces to minimize field connections and shipped directly to the San Francisco International Airport site on barges (some 26 trips back and forth).

The center three-chord trusses were shipped in temporary cradles, which acted as saddles during handling, lifting, shipping, and transporting. Pieces were transported across runways at night during off-peak hours, coordinated with airport operations. The five main center trusses were launched into position and set below final locations at the level 3 departures floor prior to erecting the steel frame, main roof box columns, and cantilevered truss pieces. Once complete, the center trusses were hydraulically jacked into position within tolerances of +/- 1" and the 6" diameter pins were placed. The relative flexibility of the tall main roof supporting steel box columns allowed for movement to achieve erection tolerances. The roof structure including roof purlins and bracing is approximately 3200 tons of steel. The center trusses weighed approximately 102 tons. Flat truss cantilever pieces weighed approximately 50 tons,

Shown above is tthe window wall frame and canopy connections

and, center flat truss pieces (cruciform to cruciform) weighed approximately 115 tons each.

Exterior Glass Wall

The Main Terminal's glass-enclosed "great hall" - 215m (705') long, 64m (210') wide, and up to 25m (83') high - creates a dramatic departure point for travelers, but does so with an economy of form and material. Vertically spanning tapered WF mullions 457mm to 914mm (18" to 36") deep are 12.2m (40') and 18.3m (60') on center on the main west, north and south facades. The east window wall structure is cantilevered from the top of the office block. The top connection to the roof structure accommodates +/-150mm (6") of vertical movement. Over the length of the west entry wall and departure level entries, hangs the front tapered canopy which spans 11m (36') out from each vertical window wall mullion as initially conceived by Myron Goldsmith.

Structural Performance Goals & Evaluation

The New International Terminal structure, responding to the architectural desire for a "great hall" enclosure with a tall glass window wall, demanded the highest performance levels. For Earthquake D (a probability exceedance of 10% in 100 years), no structural damage to the main roof and window wall were expected. Because of the low redundancy of the roof structure resting on only 20 tall-cantilevered columns and the high elevation of the glass wall, the following criteria were established for structural design in addition to the criteria of the terminal to remain operational:

- No post-elastic or yielding permitted in the cantilever columns.
- Maximum drift of top of glass wall to be less than 11".

The no-yielding criteria for the cantilever columns were derived from the fact that, if a single column yields, it could lead to the progressive collapse of the entire roof structure. The drift limitation criteria for the glass wall resulted from the industry standard for detailing glass panel enclosures for roughly 1" per floor maximum movement. Minor architectural damage was acceptable with no breakage of glass in the window wall system. The architects established a limit of 11" for the maximum relative displacement between the top and bottom of the window wall to ensure no breakage of glass. This limit is also consistent with acceptable performance drift criteria of 1% commonly used to define the operational limit.

Linear and non-linear dynamic time history analyses were performed to assess the performance of different structural schemes. Various frames, damping and isolation systems were considered in arriving at the final scheme to meet seismic performance goals and main roof/window wall requirements. Four different structural schemes were considered—moment frame with braced frame, moment frame, moment frame with fluid dampers and moment frame and braced frame scheme with seismic isolation. The linear analysis was performed on a three-dimensional model and ETABS v6.10 was used to perform time history analysis for the four structural schemes. The structural elements were modeled as elastic elements except that fluid dampers and base isolators were explicitly modeled for their nonlinearity.

The results showed that from a performance point of view in terms of roof displacement and expected damage, the moment frame and braced frame combination is preferred rather than just the moment frame. The addition of dampers reduces the base shear and expected damage but not sufficiently enough to reduce the roof displacement to less than 11". The scheme with Seismic isolation yields the best overall results in terms of reduced base shear, ductility demand, drift, and expected damage and roof displacement, thus ensuring a continued safe operation of the new terminal structure after a major earthquake. Seismic isolation provided the lowest construction cost for achieving the desired seismic performance.

Superstructure Design

The superstructure gravity and lateral system were combined in form of a 40' by 40' column grid and two-way moment resisting beams coupled with eccentric and concentric braced frames. The columns of the new terminal were cruciform shaped, fabricated with two W30 x 132 (typ) of grade 50 steel. Moment resisting girders were typically WF W27 x 94 (typ.) sections also of grade 50 steel. Spanning the moment resisting girders every 10' were gravity floor beams typically of W24 section with metal deck plus concrete construction to support floor loads. The 20 tall columns supporting the roof plate are made of 4" square box columns. These box columns were fabricated with 4" and 3" thick plates of yield strength of 46 ksi. Partial penetration weld details were adopted in the fabrication of these box columns while full penetration weld details were adopted at column splice locations.

The connections were designed with pre-Northridge moment connection details for the moment resisting frames since the joints stresses remain elastical with the base isolation for the earthquake D.

Isolation System Evaluation

Three isolation systems (friction pendulum bearings, lead-rubber bearings and high damping rubber bearings) were identified as practical isolation systems for the project. These isolation systems were evaluated for the same level of structural shear and associated cost. For the elastomeric bearings, the steel frame member sizes were increased to resist the additional eccentric gravity load moments (i.e. P-delta moments) that occur during seismic movements. Lower bound and upper bound limits on parameters affecting the design of these isolation systems were similar to the 1999 AASHTO specifications for seismic isolation design of bridges. The friction pendulum bearing isolation system was selected on a competitive bid of the super structure steel and the isolation systems. The selection of friction pendulum bearings resulted in savings of about 680 tons of structural steel as compared to that for the steel frame required for the elastomeric bearings.

The Main Terminal Building seismic base isolation system consists of 267 "friction pendulum" sliding bearings. These are steel isolators, which use the characteristics of a pendulum to lengthen the natural period of the isolated structure so as to avoid the strongest earthquake forces. The cast steel bearings include a stainless steel concave spherical surface and articulated slider. During an earthquake, the articulated slider within the bearing slides along the stainless steel concave surface, causing the supported structure to move with small pendulum motions. The movement of the slider generates a dynamic friction force that provides the required damping to absorb the energy of the earthquake. Earthquake-induced displacements occur primarily in the bearings, and lateral loads and shaking movements transmitted to the structure are greatly reduced. For the New International Terminal, the concave surface provides a 3 second isolated period and the earthquake force demands on the building is reduced by 70%. The bearings are designed to accommodate 508mm (20") of lateral displacement and dissipate energy from peak earthquake ground motions. At areas adjacent to the Main Terminal Building, seismic joints have been detailed to accommodate from 508mm to 864mm (20" to 34") of horizontal and about 50mm (2") of vertical movement.

Construction

The new terminal consists of several structures which interconnect the new terminal with existing domestic terminals, new boarding gates, approach roadways and parking structures, the ART system and a BART station. The structural systems which include approximately 30,000 tons (66,000 kips) of structural steel were developed from a basic 12.2 m square (40' x 40') column grid and floor framing module. Typical construction consists of structural steel framing with concrete composite slab deck. The foundations utilize both 305mm (12") square - 1335kN (100 ton) compression, 400kN (45 ton) tension; and, 356mm (14") square - (150 ton) compression, 489kN (55 ton) tension, precast prestressed concrete piles driven to end bearing at depths ranging from 24m (80') to 42m (140') deep. With approximately 6000 piles, they have been designed to transfer lateral loads as "fixed-ends" into cast-in-place pile caps and interconnecting grade beams. Piles were further reinforced over entire length to provide adequate ductility to withstand 244m (800') radius of curvature under earth-

Shown at top is an open view of the bearing (at left is the concave surface; at right is the articulated slider. Pictured above is an isolator over a pile cap.

quake loading.

The base isolated main terminal building superstructure lateral resisting frame consists of steel concentric and eccentric braced frames with fully welded moment connections to cruciform shaped split-tee steel W columns. Long spans of 70' and 80' over access roadways are rolled W sections. The 700' long office block rises four stories above the second and third base levels on the east. The main roof structure is supported on two sets of 10 steel box columns - 1168mm sq. (46") with 76mm-102mm (3"-4") plate thickness. At a column height of 15.7m (51.5') above Level 3 (departure level), a spherical ball joint receives the base of the cruciform verticals of the main roof trusses. The seismic isolation plane at Level 1 (baggage level) moves to Level 2 (arrival level) over the access roadway concrete walls extending up from foundation pile caps between the lanes of traffic. The friction pendulum bearings were installed above the pile caps and the access roadway concrete walls.

The systems approach of evaluation of different structural schemes for the expected performance of the New International Terminal dictated a structural scheme that utilizes seismic isolation. Seismic isolation provided the lowest construction cost for achieving the desired seismic performance. The steel seismic isolators provided the necessary strength and stability to mitigate a magnitude eight earthquake and yet deliver the desired expected performance. The seismic design of the architecturally complex New International Terminal with long spans and tall curtain walls was accomplished with the use of 267 friction pendulum seismic isolation bearings. Seismic isolation now gives the architects the tools to design complex and delicate buildings without having to fear about earthquakes, as demonstrated by the seismic isolation design of the New International Terminal.

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