# Structural Upgrade of The Boeing Commercial Airplane High Bay Assembly Plant at Everett, Washington

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**Abstract**

Choosing an appropriate upgrade system for an existing structure requires consideration of many facets of the project. This paper will discuss several of those from an engineer's viewpoint. The owner's objectives, compatibility with the existing building structure, construction feasibility, operational impact and cost are all considerations when deciding to embark upon a seismic upgrade of an existing facility.

**Introduction**

The Boeing Commercial Airplane High Bay Factory at Everett, Washington is the world's largest building by volume. It is used for the assembly of wide-bodied aircraft, Boeing 747s, 767s and 777 jetliners. It is a complex of six major building structures, with a total footprint of approximately 2350'x1600'. These structures were constructed in three phases over a 30-year period with numerous additions and modifications. The main structural system of each building consists of three 50' wide, five story towers supporting 300' or 350' clear span trusses. Clear height to the underside of these trusses is 85'. The tower areas are used for a variety of functions including stores for small parts used in assembly, storage of tooling, and offices housing various functions associated with aircraft production.

In 1996 The Boeing Company embarked upon a significant expansion of the office space housed within the tower structures of the assembly plant. This expansion added an additional 400,000-sq. ft. of new floor space in the towers, a significant addition of mass even for a structure of this magnitude. The original designs provided for the possibility of future floors in the towers based on the original design codes. Building design codes and the knowledge base regarding seismic performance of structures have changed significantly over 30 years. Because of these changes, it was decided to undertake a general structural upgrade of the buildings to accommodate the office space expansion and to incorporate a seismic upgrade of the structures.

The Boeing Company has a long relationship with The Austin Company, designer and constructor of the original complex of buildings. The Austin Company was asked to develop the construction documents for the proposed addition of new floors in the tower areas and the necessary seismic upgrades. Conventional upgrade schemes were investigated and found to require expensive and time-consuming foundation work.

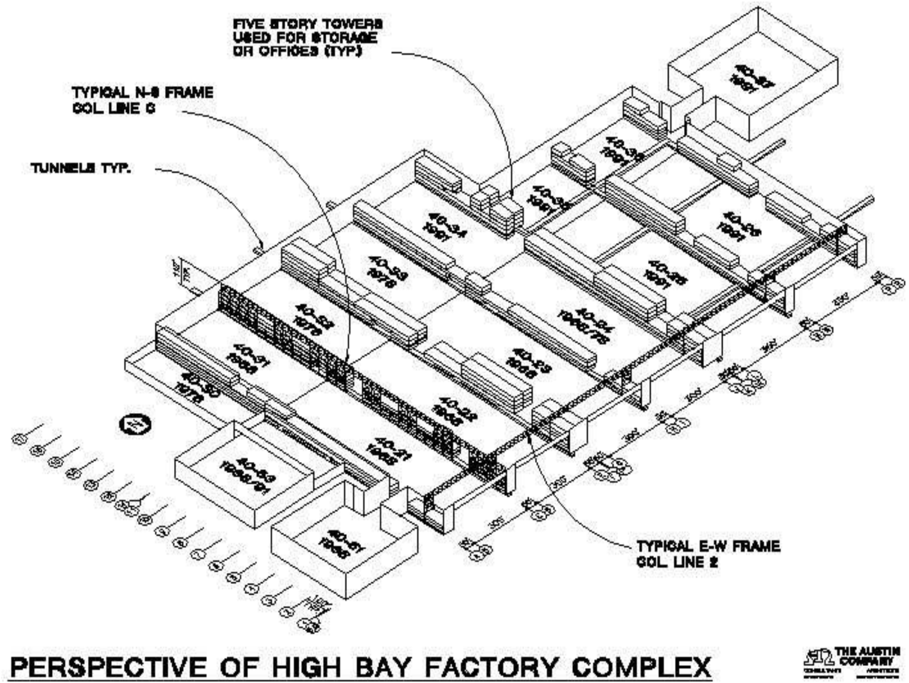



*Figure 1.*

Any foundation work would have interfered with the production activities and this motivated a search for alternatives to conventional strengthening approaches. Supplemental damping in conjunction with appropriate strengthening was chosen as the most appropriate approach for this project. The installation of Pall friction-dampers in selected bracing members reduced both internal forces and lateral deflections. Thus, strengthening of foundations was avoided. The chosen system provided significant savings in both construction cost and time.

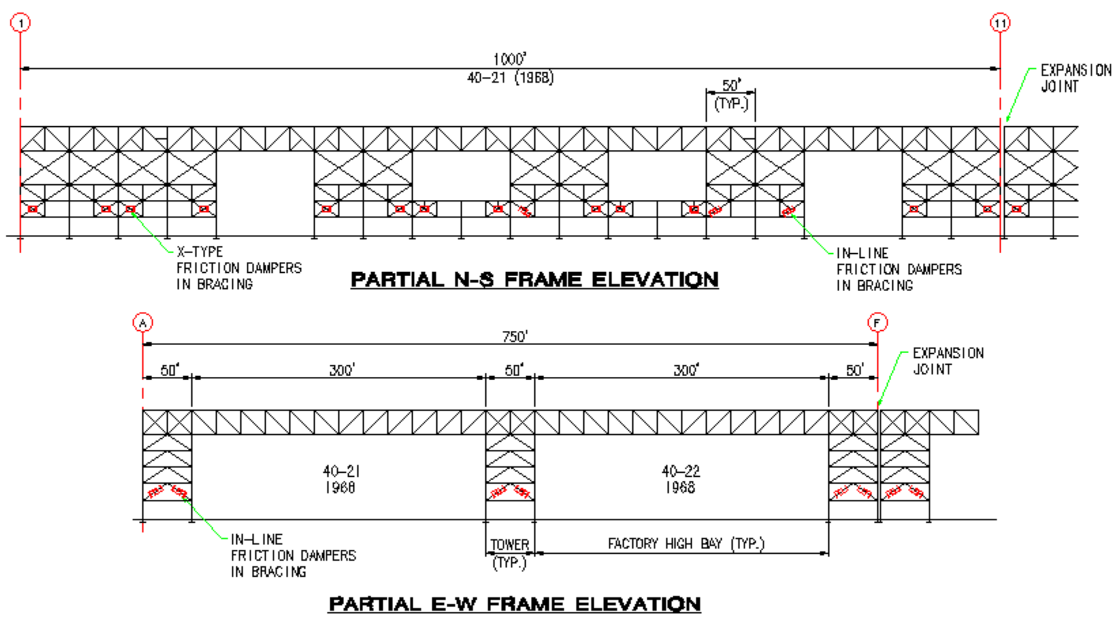



*Figure 2*

## Seismic Decision

As this project moved from planning stages to implementation, preliminary analysis of the existing structures indicated deficiencies in the existing structures for seismic load cases. These deficiencies stemmed from the use of modern analysis techniques, new codes, and an improved understanding of seismic risk and building performance.

The original buildings were designed with a bracing free ground floor level to accommodate production requirements. The first story is 20' high, which produces a significant soft story effect. Contemporary building codes consider this a vulnerable condition. In addition, many areas in the towers were braced with tension-only bracing; another practice that now is known to perform poorly during a seismic event. The additional mass being added to the buildings, in some towers as much as doubling the existing floor mass, compounded the initial structural deficiencies. Because of these issues, The Austin Company recommended that a seismic upgrade would be prudent.

Engineering and construction of the additional elevated office tower floors continued with the seismic upgrade to follow as the design progressed. Floor construction continued as the seismic team was put in place, and preliminary studies were made to identify a workable solution.

## Studies

Several preliminary studies were conducted to identify a viable solution to the problem of how to economically implement a seismic upgrade of the largest structure in the world without interfering with aircraft production. The requirement to minimize or eliminate grade level activities as well as aerial work in the overhead crane space in order to minimize impact on aircraft production was a critical concern for Boeing. Also, the main ground transportation aisles in the N-S direction are located within the footprint of each tower. These conditions virtually eliminated the possibility of adding bracing and/or shear walls to the ground to eliminate the soft story effect. The main thrust of these studies resulted in a comparison of the "brute strength" method of reinforcement and the use of friction dampers to dissipate seismic energy.

## Conventional brute strength approach

The first story columns were identified as the primary weak link in the structure's ability to resist seismic motion. The obvious solution consisted of strengthening the columns by the addition of heavy cover plates. Conventional linear elastic static analysis techniques were used in the analysis. Using this approach, the heavy column cover plates increased the building stiffness. This, in turn, increased the building's base shear. The higher base shear required increasing the cover plate size, and so on, until the base shear far exceeded the original design base shear. This meant strengthening or replacing virtually all bracing, and improving connections with the addition of tie plates to transfer axial forces through the columns. Significant foundation work would also be necessary to develop the increased base shear.

As this approach would require essentially rebuilding a majority of the structure in-place, Boeing requested alternative approaches be investigated, and invited Casper, Phillips & Associates (CP&A) to the team. CP&A verified The Austin Company's conclusions regarding the adequacy of the existing design, and suggested several alternatives to column cover plates. Unfortunately, all conventional approaches led to the same overall conclusion – stiffening the building meant replacing most of the existing bracing and modifying most of the existing connections. Back to the drawing board!

**The non-linear approach…**

CP&A had been working for years with Prof. Ed Wilson, of the University of California. Dr. Wilson had recently implemented his "fast dynamic method" for non-linear time-history analysis in his personal version of the SAP finite element analysis program. This analysis technique was being used on the seismic retrofit of large bridges in California. Dr. Wilson worked with CP&A to study the use of this method as an alternative to conventional elastic methods. Since this non-linear method predicts actual forces and displacements for a given ground motion, it was an ideal tool for realistic evaluation of the seismic performance of the existing building and any proposed upgrade schemes. The primary concern was to achieve seismic performance consistent with the intent of the 1994 UBC.

Several models of the existing structure were made and subjected to a stock standard earthquake ground motion (El Centro, CA). The results were quite informative – while the member forces were relatively low, the lateral displacements were excessive.

**…plus friction dampers**

At The Boeing Company's suggestion a relatively new concept was explored; friction dampers were introduced into the model. Initially, the dampers were only placed between buildings, near the roof. The results looked promising, with reduced base shear and displacement. However, due to the limited number of damper locations, large dampers were required which necessitated significant local strengthening. Several other configurations of dampers were modeled. Finally, dampers were placed in the bracing between the second and third floor (Figures 6 and 7). The second floor damped braces act as a transition between the soft first story and the braced frames above. With the dampers spread throughout the building in both directions, the resulting analysis showed significantly lower base shear, displacement and member forces when compared to the brute force stiffening approach. Only selected bracing members required strengthening or replacement. Most beam to column connections required strengthening for the transfer of axial forces.

A comparative upgrade design was carried out using both methods. The results indicated a substantial saving in construction costs with the use of dampers. Table 1 indicates the qualitative results of this comparison.

**Table 1.**

| Solution Method | Brute Strength | Energy Dissipation |
|---|---|---|
| **Building Stiffness** | Increase | Decrease |
| **Column Moments** | Increase | Decrease |
| **Brace Forces** | Increase | Decrease |
| **Deflection** | Minor Decrease | Decrease |
| **Base Shear** | Increase | Decrease |
| **Const. Difficulty** | High | Moderate |
| **Cost Estimate** | High | Moderate |
| **Risk Reduction** | Moderate | High |
| **Production Impact** | High | Low |

The use of friction dampers resulted in much lower internal forces being developed in the structure, as well as lower deflections. This effectively reduced the quantity and extent of existing members and connections requiring modification, thus reducing construction schedule and costs. Column strengthening and foundation work was eliminated, and interruption of production activities was minimal.

## Seismic Criteria

Once the decision for a damper-based seismic upgrade had been made, a team of engineers and consultants was assembled. This team consisted of: The Austin Company, engineer of record for the project, Casper Philips and Associates, structural consultants to assist in the analysis, and Dames and Moore, soils and seismic consultants. The peer reviewer, Degenkolb Engineers, was added to the team early on to validate the preliminary study results and endorse the design criteria to be used. Through a series of meetings held over the course of several months, the analysis, acceptance and design criteria were developed.

The intent of the design criteria was to provide for a design that would result in a building structure with performance characteristics consistent with the intent of the 1994 Uniform Building Code (UBC) which was in effect at the time. However, the UBC does not adequately address the design and retrofit of existing structures using passive energy dissipation devices. FEMA-273 / September 1996 was thus adopted as the guideline basis for the design criteria for the structural seismic upgrade. Since FEMA-273 was in ballot form and had not been adopted, full "compliance" with FEMA-273 was not intended, expected or even possible. The criteria used for the seismic upgrade were adapted from FEMA-273. However, the adopted criteria were interpreted, expanded, or modified from FEMA-273 as necessary for the particular goals and methodology used for this project. For instance, Table 2 shows the differing seismic event probabilities recommended by the UBC and FEMA-273.

**Table 2.**

| Code | Seismic Event | Probability of being Exceeded |
|---|---|---|
| FEMA-273 Sept. 96 | Basic Safety Earthquake 1 (BSE-1) | 10% in 50 years |
| FEMA-273 Sept. 96 | Basic Safety Earthquake 2 (BSE-2) | 2% in 50 years. |
| UBC-1994 | Design Basis Earthquake (DBE) | 10% in 50 years |
| UBC-1994 | Maximum Capable Earthquake (MCE) | 10% in 100 years |

Boeing's seismological experts, Dames & Moore, recommended using the UBC's MCE as the definition for BSE-2. The performance levels of the building structure for each ground motion were established consistent with UBC-1994 and FEMA-273.

The Boeing Company will only add additional floors to a portion of the existing towers at this time. They wanted the ability to add additional floors in the future without having to address the seismic issues again. For the correct performance of the modified structural system dampers were required to be distributed throughout the factory. This meant upgrading all areas of the factory even where no new floors were planned at the present. Preliminary analysis with the dampened building model indicated that in some instances the "existing", partial build-out, configuration of the structures controlled member or connection design. In other cases member design was controlled by the "full" future build-out condition. Therefore it was determined that both the "existing" and the "full" build-out conditions would be investigated for the final design.

## Analysis

The use of energy dissipation devices required the use of nonlinear time-history dynamic analysis. The analysis was carried out using the FEA program SAP2000-Nonlinear, a commercial version of Professor Wilson's SAP program. Six different earthquake analyses were run (both horizontal components for each of the 3 specified earthquakes) for each model at BSE-1 and BSE-2 levels.

P-delta effects were included. Where applicable, plastic hinges in columns were modeled. The dynamic stiffness K of the hinge was set to 1000 times the stiffness of the column (viewed as chord rotation). The effective (or elastic) stiffness of the hinge was set to K. The "yield" moment was set to the full plastic moment reduced by the axial force. Strain hardening was ignored due to lack of significant rotation.

Friction dampers were modeled as nonlinear plastic elements. Since the hysteretic behavior of a friction damper is similar to an ideal elasto-plastic material, the modeling was very simple - the slip load of a friction damper is modeled as a fictitious yield force.

Generally, braces were modeled as pin-pin, unless significant connection stiffness existed. Braces that would buckle were modeled as two elements, a linear element that accounted for the true elastic behavior in series with a non-linear gap element. The gap element had K = 1000AE/L, plus a pretension load equal to the calculated buckling load. Braces that yield in tension and buckle in compression were modeled with two non-linear elements, a yielding element in series with a pre-tensioned gap element. Thermal loads were used to model fabricated truss camber.

Modal damping of 1% was used for all modes and the time step for output was set to 0.02 seconds. Ritz vectors (mode shapes) were used in lieu of Eigenvectors, consistent with the fast dynamic method.

After several iterations, the optimum slip load of the friction dampers was established to achieve minimum seismic response. A total of 537 friction dampers, varying in capacity from 75kips to 200kips and with slip lengths of ?6 to ?7 inches, were used to achieve the desired response. The slip loads were set high enough to prevent movement from wind loads.

The energy plot (Figure 3, below) shows that the friction dampers have dissipated about 60% of the total energy input. This has resulted in significant reduction in response.

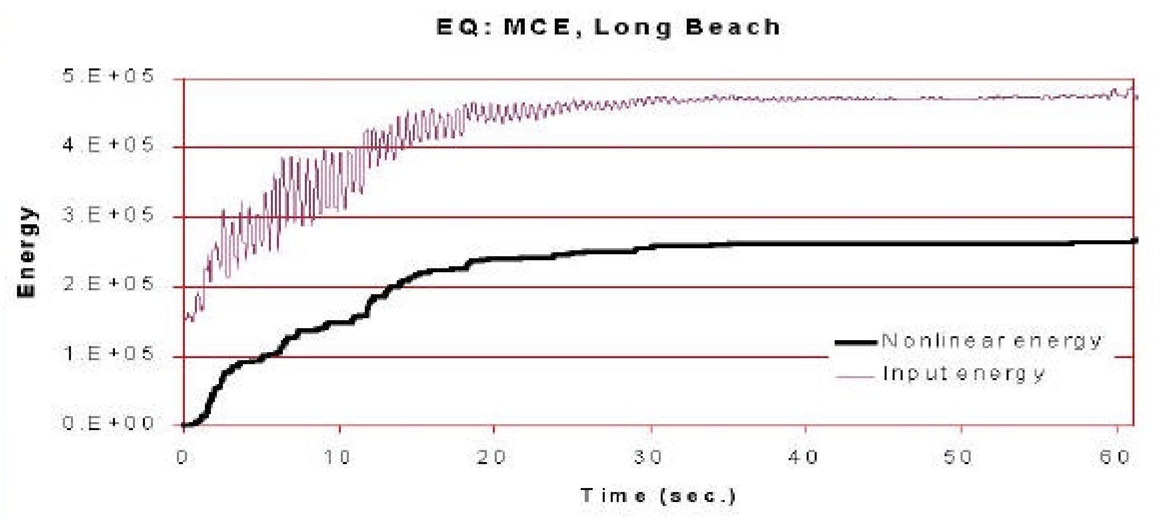



*Figure 3. Chart Showing Total Energy Input and Energy Dissipated by Friction Dampers*

A time-history plot (Figure 4) shows the lateral deflection at the top of the building for one of the MCE earthquake records. The maximum amplitude is 9 inches (drift 0.67 %). After the earthquake, the structure returns almost to its original position.

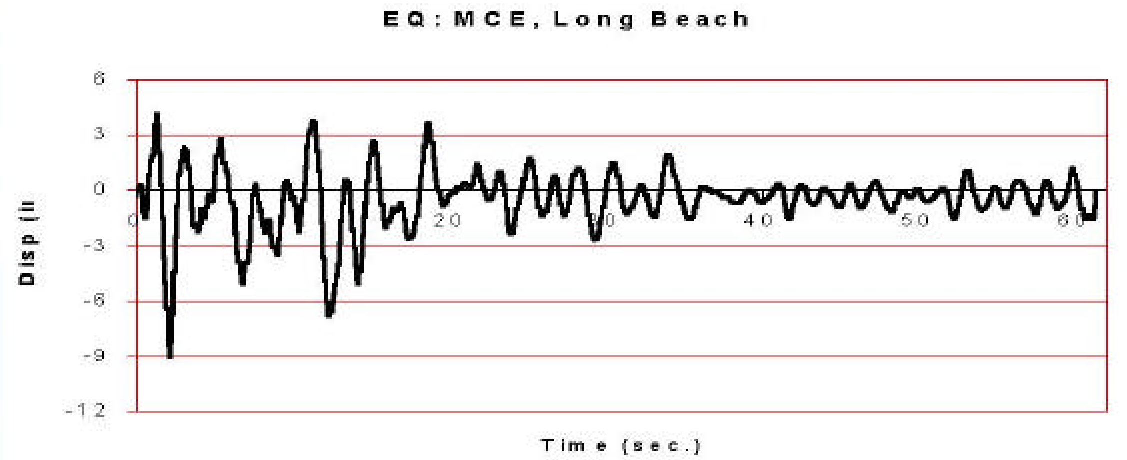



*Figure 4. Typical Time History of Displacement at Roof.*

A typical hysteresis loop of a friction damper is shown in Figure 5, below. Generally, the slip lengths for the dampers were calculated by multiplying the largest slip values for BSE-2 by 1.3.

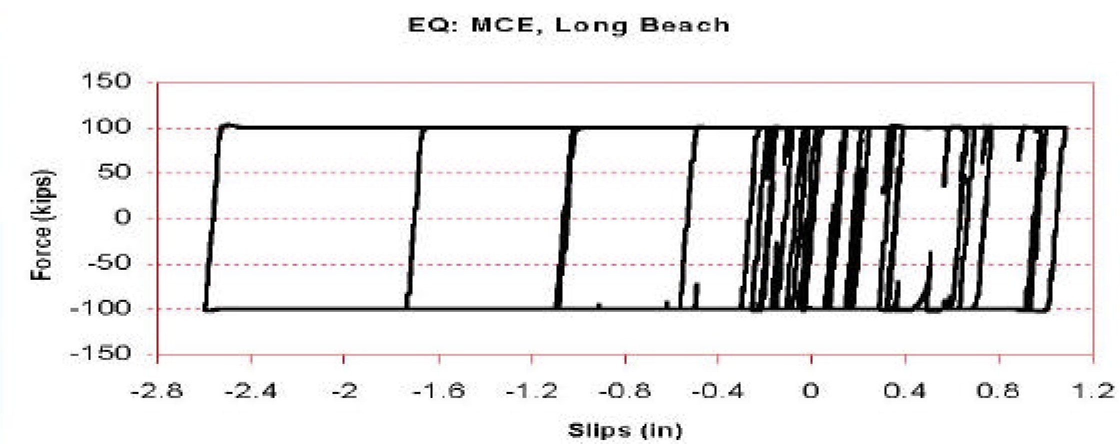



*Figure 5. Hysteresis Loop of a Typical Friction Damper*

**Design**

Analysis with the use of time histories produced massive amounts of data. SAP2000 was capable of calculating maximum envelope forces and making member checks based on these envelope forces. This is conservative, and usually desirable for new structures. But this approach is too conservative for checking existing members with the hope of minimizing the amount of replacement or reinforcement required. CP&A developed a post-processing program (called THAnalysis) to process the time-history data and evaluate the design checks at each time step, thus minimizing the amount of strengthening required.

Once the time history results were determined using SAP2000N, THAnalysis directly accessed the results in their native binary format, ensuring no loss of precision. It used engineer-defined design check algorithms to evaluate members and connections.

All member and joint design checks were made using the exact time-step forces. With this approach, many of the existing inverted chevron braces in the E-W frames needed no strengthening. All of the tension only bracing in the N-S frames needed to be replaced with new tube braces, capable of taking compression as well as tension. The members to which the Pall Friction Dampers are attached have forces limited by the damper capacity and therefore, did not require replacement.

**Existing Structure**

The existing structure, having been designed and constructed in three major phases, consists of three distinct sets of construction details. All buildings share a common structural concept. Long span trusses spanning between towers braced to the second floor level. Secondary roof trusses span parallel to the towers at 25-foot spacing. Roof deck diaphragm and bottom chord horizontal bracing tie the towers together and transfer lateral forces to the bracing system. Main column spacing in the tower area is 100 feet in the N-S direction with intermediate columns at 50 feet supporting the floor framing. The towers are braced in both directions from the second level to the roof, providing a relatively rigid structure above the second floor.

The original set of structures constructed in 1968 utilizes columns made up of heavy W14 sections and plates, creating a box shape. 300-foot long trusses span between towers constructed of heavy W shapes. A true pin connection was utilized at the top chord/column connection providing for a simple span truss. Floor framing consists of W24 girders supporting W24 composite floor beams. All floor framing connections were designed as simple shear connections utilizing shear tabs or double angles. The girders span 50 feet between columns with the vertical bracing system intended to provide support for the floor girder at the mid point. The bracing members were double angle tension only design in the N-S frames and W shape inverted chevron bracing in the E-W frames.

A major expansion was constructed in 1978. This followed the general formula of the original structures. However, code revisions forced a change to much larger star column sections made up of W36 members and 2 W14's welded to create a star shape. Girder and beam to column connections utilized end plates bolted to the column flanges. Again, vertical bracing consisted mainly of double angles, somewhat heavier, in the N-S direction and W shapes in the E-W frames.

The most recent expansion occurred in 1991. Here a new concept was utilized. The 350-foot long roof trusses were designed as continuous across the towers. Vertical tension only bracing was not used. Instead, eccentric braced frames were used in both the N-S and E-W frame directions. The soft first story was maintained, but recognized as a weakness and treated as such. Column sections took a large jump in size and floor framing was designed to carry gravity loads without support from the vertical bracing.

Generally, a review of analysis results indicated a significant number of bracing members and all major beam to column connections in the 1968 construction required strengthening or replacing. Fewer members, but still the majority of beam column connections in the 1978 construction require attention. And as expected the 1991 areas require little in the way of improvement. In addition to bracing and connection improvements, it was found that the intermediate tower columns of the 1968 construction required some minor strengthening between the ground and the second level due to P-? effects.

**Pall Friction Dampers**

The most visible components of this design are the Pall friction-dampers. 537 dampers were required for this project varying in capacity from 75kips to 200kips with a slip length of +/-6 inches to +/-7 inches. In-line damper installation details varied due to the variety of member profiles and bracing connections associated with the existing structures. Typically the existing bracing member was cut near the lower end connection to facilitate access. Installation details were developed with the intent to provide as close to a pin connection at the end of the damper as practical to minimize bending induced in the damper assembly. Out of plane stability was also addressed when the connection details were developed. The new Pall friction-damper was then installed using the appropriate detail. Figure 6 is typical of an in-line damper installation. Coordination of end connection details with damper fabrication was necessary to accommodate detail variations.

The Pall X-type damper (Figure 7) allowed the reuse of tension only braces at a number of locations. These dampers allow the tension leg of an X-brace to take force while preventing the compression leg from effectively sharing this load. This eliminates the buckling of slender compression legs, which have proved detrimental to cyclic load capacities.

*Figure 6. In-line Pall Friction Damper*

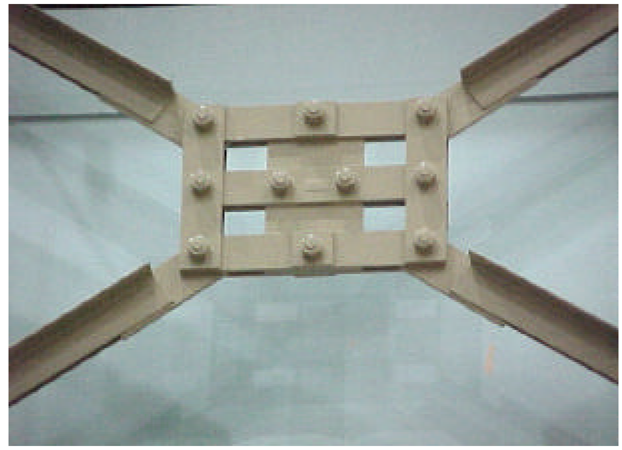


*Figure 7. Cross Brace Pall Friction Damper*

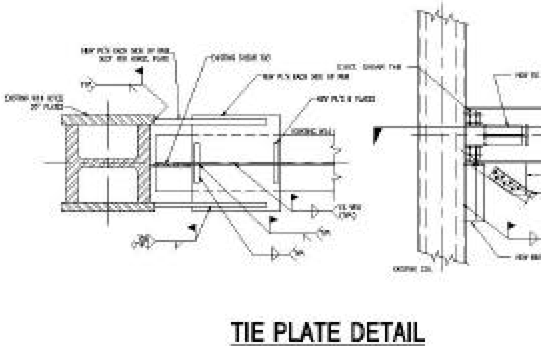
Due to construction considerations, where existing bracing members were not qualified it was decided to replace rather than strengthen these members. The use of lead based paint on all early construction necessitated costly and time-consuming procedures to mitigate. Therefore, designs were developed to minimize these procedures. Replacing members rather than welding strengthening plates became more economical except in areas of difficult access where the installation of 35-foot long members became impossible.

Connection details differ greatly from one phase of construction to another. Therefore the new seismic details reflect these differences. Early building columns were box shaped. Welded tie plates were designed to transfer forces from the horizontal-framing members into the columns at the simple shear connections. These were sized based on the forces developed from the time-history analysis using the agreed upon criteria. An attempt was made to keep these plates located near the centroid of the member to minimize bending on the plates and create a pure axial tie. Figure 8 depicts typical connection details at 1968 construction.

In the event of a major earthquake, severe enough to cause joint rotations capable of destroying the shear tab connection to the column, new beam seats were added at most beam to column connections. These provide the safety against collapse prevention at full design magnitude seismic events.

The 1978 construction columns were of a star shape configuration. This necessitated a revised detail for transfer of axial forces. Rather than welding plates to the beam and column, a detail utilizing threaded rods (Figure 9) was developed. This solution gave the added benefit of eliminating much of the field welding associated with the plate detail.

The transfer of axial force through the beam to column connection in the 1991 construction was simplified through the fact that this construction utilized shear tabs for both beam and bracing connections. A second shear tab is welded to the column on the opposite side of the beam web at the existing tab. Bolts are replaced with longer bolts of the same diameter now utilized in double shear.



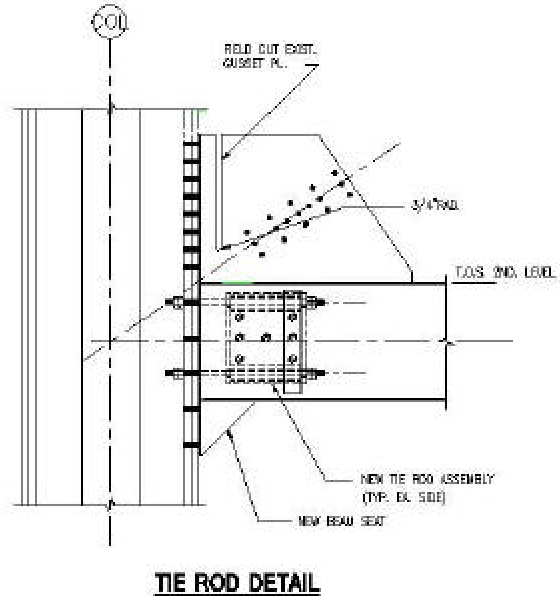




*Figure 8* *Figure 9*

Main truss and roof truss to column connections are another area of concern. The 1968 construction details made use of very thin plates at the top chord connections. When subjected to the high axial forces generated by dynamic loads, these plates proved inadequate. Fortunately, the details lent themselves to a relatively simple solution. Rods are used to transfer the axial forces at the top chord level through the column. Similar details are used at the bottom chord in the N-S frames (Figure 10).

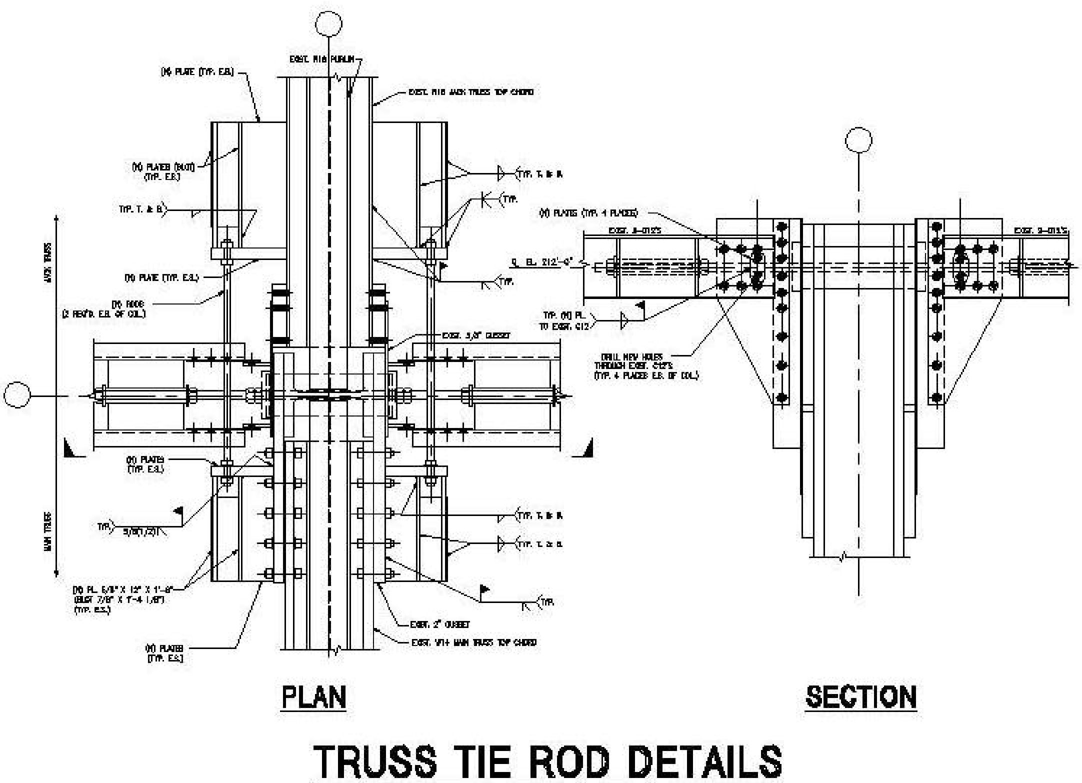



*Figure 10*

Truss connections in the 1978 construction were similar in concept to those of the 1968 construction. However, the connecting plates used were much heavier, and therefore, the extent of strengthening required is substantially less for the 1978 structures. The E-W main truss strengthening was not required for the 1978 construction, simplifying this construction significantly.

## Construction

Retrofit of an existing facility is never easy, even under ideal circumstances. The prime directive for construction for this project was "aircraft assembly schedules shall not be impacted." This has made the coordination between construction and The Boeing Company's production personnel critical. The Boeing Company Construction Management Team has very capably handled the interface between construction and production personnel.

Careful planning and open relations with the effected production personnel have minimized problems. As construction prepares to enter an area, a detailed construction plan is prepared. This is discussed with the personnel in the effected area, and coordination of construction activities to minimize interference is paramount. The production staff knows every detail concerning their disruption. Each temporary move is planned and coordinated well in advance so there are no surprises.

Construction commenced prior to completion of the engineering and is scheduled to continue through 2002. Engineering has worked closely with the field forces to minimize the impact of the construction on aircraft production and to simplify construction details. As the project has progressed many suggestions to simplify construction have been incorporated into the engineering design.

To assist construction in the interpretation of the design documents, as well as provide engineering with a "pair of eyes" on site to aid in understanding and dealing with construction concerns, a full time engineer has been located on site. This person is responsible for the accurate conveyance to engineering of problems encountered in the field and suggests possible solutions that can be constructed. These problems and their solutions are handled formally in an RFI, request for information, process. Many of these problems, such as minor utility interferences or access problems can be dealt with immediately by the site engineer with little project delay. Major issues are clarified by the site engineer in a formal RFI and submitted to office engineering. Solutions are achieved through consensus among office engineering, construction, and the site engineer.

## Conclusion

Projects such as this come to an engineer once in a lifetime. The chance to participate in a project of this complexity and magnitude is rare. Applying new technologies on this scale was a challenge and a great learning opportunity for all the participants.

## Acknowledgements

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