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SEISMIC UPGRADE OF THE BOEING COMMERCIAL AIRPLANE FACTORY AT EVERETT, WA, USA

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SUMMARY

In 1996, the seismic upgrade of existing structure was undertaken along with other major renovations. The conventional seismic upgrade schemes required expensive and time-consuming work on foundations. Supplemental damping in conjunction with appropriate stiffness was considered to be the best solution for this structure. Nonlinear time-history dynamic analysis was used to simulate actual building performance and to give realistic design forces. The use of Pall Friction Dampers significantly reduced the forces exerted and lateral deflections. Therefore, the strengthening of existing members was minimized and the strengthening of foundations was avoided. The chosen scheme provided significant saving in both construction cost and construction time.

INTRODUCTION

The Boeing Commercial Airplane Factory at Everett is the world's largest building by volume, and could contain Disneyland under one roof. It is used for the assembly of wide-bodied aircraft, Boeing 747, 767 and 777 jetliners. It is a complex of six major building structures, with a total footprint of approximately 2350'x1600' (Figures 1-4). These structures were constructed in three phases over a 30-year period with many additions and modifications. The main structural system of each building consists of three 50' wide, five story towers supporting two 300' or 350' clear span and 40' deep trusses. Clear height to the underside of these trusses is 87' and the height at roof is 127'. 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. 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. Today's building codes consider this a vulnerable condition.

In 1996 Boeing embarked upon a significant expansion of the office space housed in the tower structures of the assembly plant. This expansion added an additional 400,000-sq. ft. of new floor space in the towers. The additional mass being added to the buildings compounded the initial structural deficiencies.

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The building codes have changed significantly over a period of 30 years, especially in respect of seismic design. Therefore, it was decided to undertake a seismic upgrade along with general structural upgrade of the buildings to accommodate the office space expansion.

Figure 1. Interior View of the Factory

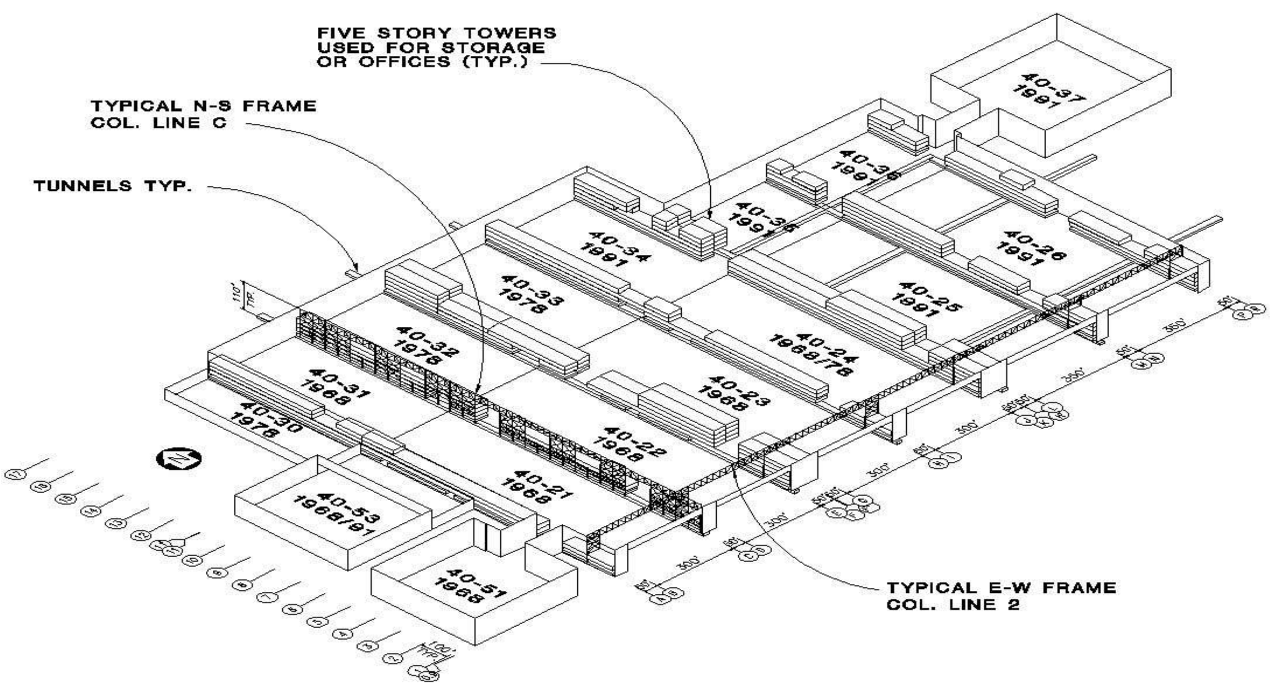

Figure 2. Perspective View of High Bay Building Complex

The Austin Company, who designed and constructed the original complex of buildings, was given the task of developing the construction documents for the proposed additions. A general review of preliminary analysis indicated that a significant number of bracing members and all major beam-column connections of 1968 construction required strengthening or replacing. Fewer members, but still the majority of beam column connections in the 1978 construction required attention. 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 soft story effects. The Austin Co. recommended that a seismic upgrade would be prudent.

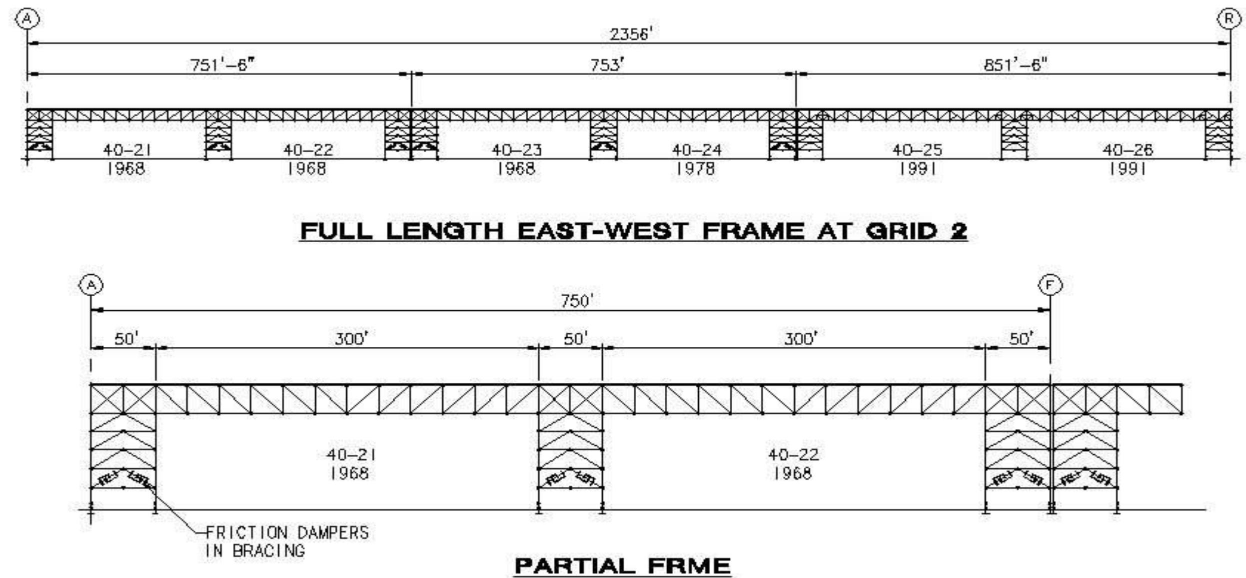

Figure 3. Typical East-West Frames

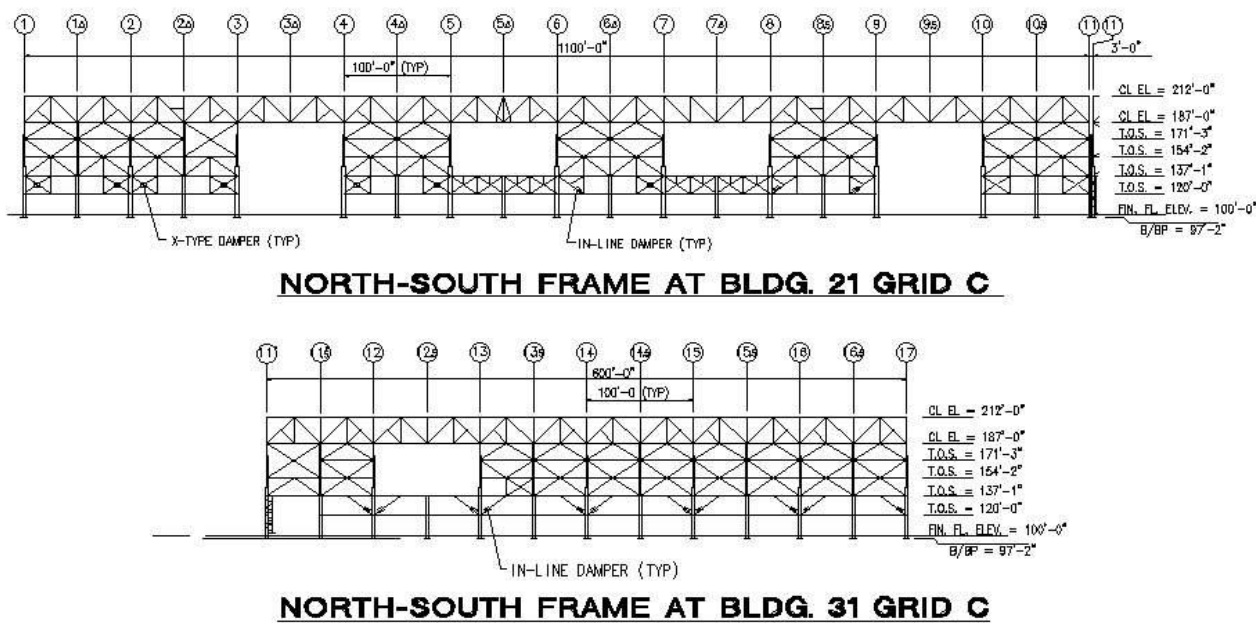

Figure 4. Typical North-South Frames

DESCRIPTION OF EXISTING STRUCTURE

The existing structure has been designed and constructed in three major phases. These consist 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 framing. The towers are braced in both directions from the 2nd level to the roof providing a relatively rigid structure above the 2nd floor.

The structures of buildings 40-21, 22, 23, and 31, constructed in 1968, utilize columns made up of heavy W14 sections and plates creating a box shape. 300-350 feet 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 beam 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.

The second major expansion was in 1978. In general, this followed the original structures. However, code revisions necessitated a much larger star column sections made up of W36 members and 2 W14's. 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 last 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 again took a large jump in size and floor framing was designed to carry gravity loads without support from vertical bracing.

SELECTION OF SCHEME FOR SEISMIC UPGRADE

Several preliminary studies were conducted to identify a viable solution to the problem of how to economically implement a seismic upgrade of the structure without interfering with aircraft production. The requirement to minimize or eliminate grade level activities 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 plan of each tower. These conditions virtually eliminated the possibility of adding bracing or shear walls at the ground to eliminate the soft story effect.

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 the braces, and improving connections with the addition of tie plates to transfer axial forces through the columns.

As this approach would require essentially rebuilding a majority of the structure, Boeing requested alternative approaches be investigated, and invited Casper, Phillips & Associates (CPA) to the team. CPA verified The Austin Co.'s conclusions regarding the adequacy of the existing design, and suggested several alternatives to column cover plates. Unfortunately, all conventional approaches lead to the same overall conclusion – stiffening the building meant replacing most of the existing bracing and modifying most of the existing connections. Also, the conventional upgrade schemes required expensive and time consuming foundation work. Any foundation work would have interfered with the production activities. This motivated a search for alternatives to conventional strengthening approaches.

Supplemental Damping Approach.
Introduction of supplemental damping was considered to be the most appropriate approach for the seismic upgrade of this project. Since dampers dissipate most of the seismic energy, the forces exerted on the structure and the deflections are considerably reduced. Thus, strengthening of several members and connections can be minimized and expensive foundation work could be avoided.

The stability considerations dictated that the energy dissipating system, when incorporated in existing bracing, should possess sufficient stiffness. The dampers should be easily adaptable to all types of bracing, including tension-only cross bracing. As Boeing could not afford interruption to their aircraft production, it was important that the chosen system requires the least inspection or maintenance. Besides, cost of dampers, construction cost and construction time, were important considerations.

The above criteria lead to the selection of Pall Friction Dampers for the seismic upgrade.

PALL FRICTION DAMPERS

Pall Friction Dampers are simple and foolproof in construction and inexpensive in cost, Pall [5-7]. These dampers have successfully gone through rigorous proof testing on shake tables in Canada and the United States. In 1987, a nine story three bay frame, equipped with friction dampers, was successfully proof tested on a shake table at Earthquake Engineering Research Center of the University of California at Berkeley, Kelly [8]. Even for excitations of more than 5 times of 1985 Mexican earthquake, the structure with Pall Friction Dampers remained elastic without damage.

Pall Friction Dampers possess large rectangular hysteresis loops, similar to an ideal elasto-plastic behavior, with negligible fade over several cycles of reversals. The performance of Pall Friction Dampers is independent of velocity. For a given force and displacement in a damper, the energy dissipation of Pall Friction Damper is the largest compared to other damping devices. Therefore, fewer Pall Friction Dampers are required to provide a given amount of supplemental damping. These dampers also provide stiffness for added stability. The maximum force in a friction damper is well defined and remains constant for any future ground motions (DBE or MCE). By choosing an appropriate slip load of friction dampers, the forces on the members can be controlled to be within their capacity. This is an engineered solution. Hence, the design of bracing and connections becomes economical. The friction dampers do not slip during wind. During severe seismic excitations, these slip at a predetermined optimum load. As there is nothing to damage or leak, they do not need regular inspection or maintenance. These dampers have found several applications in both new and retrofit of existing buildings. More than 80 buildings have already been built. For more details refer www.palldynamics.com.

Optimum Location of Friction Dampers
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 capacity dampers were required which necessitated significant local strengthening. Several other configurations of dampers were tried. Finally, dampers were placed in the bracing between the second and third floor. The second floor damped braces act as a transition between the soft first story and rigid

bracing 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 strength approach. Only few bracing members required strengthening or replacement. Most beam to column connections required strengthening for the transfer of axial forces. Column strengthening and foundation work was eliminated, and interruption of production activities was minimal.

COMPARISON OF RESULTS TO TWO APPROACHES

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

Table 1. Qualitative Comparison of Alternate Systems

Item	*Brute Strength*	*Supplemental Damping*
Building Stiffness	Increase	Decrease
Column Moments	Increase	Decrease
Brace Forces	Increase	Decrease
Deflection	Minor Decrease	Decrease
Construction Difficulty	High	Moderate
Construction Cost	High	Moderate
Risk Reduction	Moderate	High
Production Impact	High	Low

DESIGN CRITERIA

Once the decision for a 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 Phillips and Associates, structural consultants to assist in the analysis, and Dames & 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.

The 1994 Uniform Building Code (UBC) 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 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, but were interpreted, expanded, or deviated from FEMA 273 as necessary for the particular goals and methodology used for this project. 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 UBC-1994.

First, the levels of ground motions for design purposes were established. Table 2 describes various “levels” of ground motion as defined by the FEMA- 273. The UBC ground motions are noted to show that the FEMA-273 “levels” of ground motion as implemented in these criteria are consistent with the UBC-1994.

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

Table 2. Various Levels of Ground Motion

Code	*Seismic Event*	*Probability*
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 intended to build some of the tower structures in the first phase with an option to build the remainder of office upgrade in future without re-doing the seismic upgrade. All the friction dampers have to be installed in the first phase of upgrade. The performance of the friction dampers, local members, and connections is dependent upon the strength and stiffness contributions of the various frame elements. There are many locations where the “full build-out” frame elements do not yet exist. The analysis showed that governing force levels are generated locally in members for the “existing” condition (i.e. locally, force levels decrease in certain “existing” members and connections with the addition of the future build-out members and loading). As such, it was necessary to investigate the behavior and performance of both the “full build-out” and the “existing” structures.

ANALYSIS

The use of energy dissipation devices requires the use of nonlinear time-history dynamic analysis. The analysis was carried out using SAP2000 Nonlinear Program. Six different earthquake analyses were run for each of the 3 specified earthquakes for BSE-1 and BSE-2.

P-delta effects were taken into account. Where applicable, plastic hinges in columns were included in the model. The dynamic stiffness K of the hinge was set to 1.0E3 times the stiffness of the column (viewed as chord rotation). The effective (or elastic) stiffness of the hinge KE is set to K. The “yield” moment is set to the full plastic moment MPC reduced by the axial force. To account for a curved transition from the elastic yield moment to the full plastic moment (when looking at the M-θ diagram), an EXP value of 3 was used. Strain hardening was ignored due to lack of significant rotation.

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

Generally, braces were modeled as pin-pin, unless significant connection stiffness existed. Braces that 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 = 1.0E3*AE/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. The SAP2000 does not report body forces (i.e. uniform loads, thermal loads, concentrated loads along the length of a member, etc.) when outputting the dynamic beam forces at “station” points. These forces were included manually when checking members.

Modal damping of 1% was used for all modes and time step was 0.02 second. Ritz vectors (mode shapes) were used in lieu of Eigenvectors.

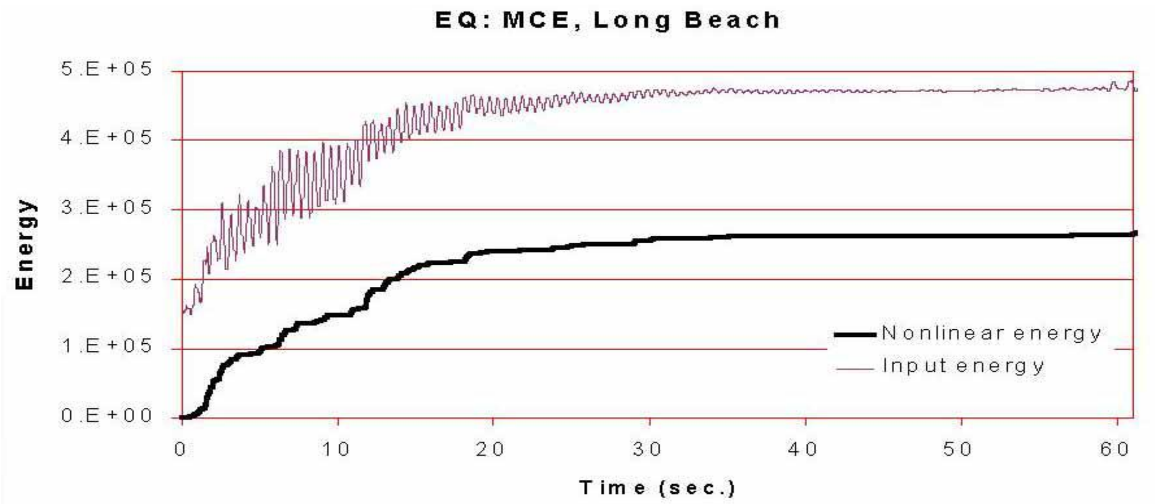

Figure 5. Total Energy Input and Energy Dissipated by Friction Dampers

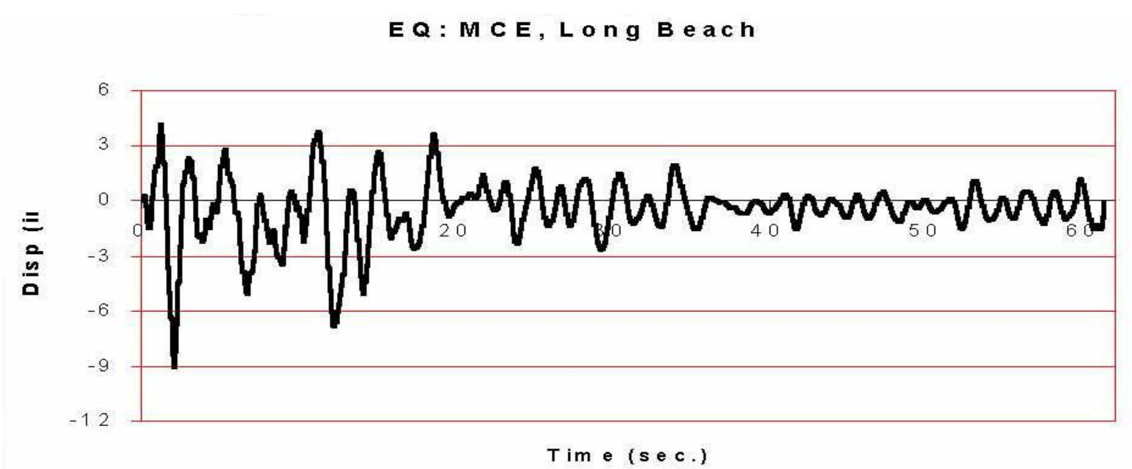

Figure 6. Typical Time History of Displacement at Roof.

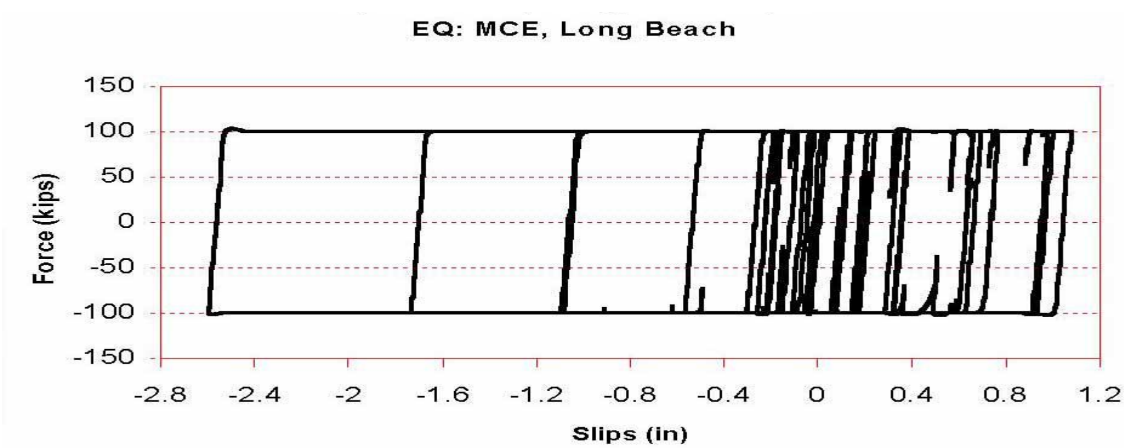

Figure 7. Hysteresis Loop of a Typical Friction Damper

With several iterations, the optimum slip load of friction dampers was established to achieve minimum seismic response. A total of 537 friction dampers, varying in capacity from 75kips to 200kips were used to achieve the desired response. As the friction dampers are to be activated only during a major earthquake, it was made sure that the slip load of damper exceeds the combined gravity and wind load.

The energy plot is shown in Figure 5. It is seen that friction dampers have dissipated about 60% of the total energy input. This has resulted in significant reduction in overall response.

Figure 6 shows time histories of deflections at top for MCE earthquake record. The maximum amplitude is 9 inches (drift 0.67 %). After the earthquake, the structure almost returns to its original position.

Typical hysteresis loop of a friction damper is shown in Figure 7. The friction dampers were designed to accommodate slip lengths of 1.3 times the largest slip values for BSE-2. The slip lengths of ±6 to ±7.5 inches were used.

DESIGN

The dynamic analysis with various time histories produced massive amount of data. SAP2000 was capable of combining maximum stresses to do member checks. This is a conservative and acceptable for new structures. In order to do minimum amount of replacement or reinforcement to existing members, this is not a desirable approach. Therefore, a special post-processing program called 'THAnalysis' was developed by CP&A to process the time-history data to find the true design case and thus minimize the amount of strengthening required.

Once the time history results were determined using SAP2000N, THAnalysis directly accessed the results in their native binary format. The THAnalysis program has several important key features:

1. Allows direct access to single or simultaneous multiple member force data.
2. Allows key geometry and properties to be associated with a single or group of members.
3. Allows the user to define variables.
4. Allows the user to use trigonometric, exponential, and other higher-math functions.
5. Allows the user to define logical expressions.
6. Allows the user to define if-then-else constructs.
7. Allows the user to enter design algorithms and equations directly.
8. Will evaluate design algorithms for envelope values or for each time step.
9. Allows the user to dictate the format and content of the output.
10. Allows the user to specify which variables to track for maximum and minimum values.

Once the design algorithms are developed for a particular joint or member type, they can be easily re-used for similar types by parameterizing the key values and assigning the actual values to a look-up table. These features were combined into a control file, so that the analysis and design of the entire structure was completely automated.

All member and joint design checks were made using the exact time-step feature. 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 is being 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.

The most visible components of this design are the Pall Friction Dampers. In-line friction dampers 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. Figure 8 is typical of an in-line Pall Friction Damper installation. Coordination of end connection details with damper fabrication was necessary to accommodate detail variations.

The Pall Friction Damper for cross bracing, Figure 9, allowed the reuse of tension-only braces at a number of locations. When one of the brace in tension reaches the slip capacity, the damper slips. The four linkages of the friction damper force the other brace to shorten and thus prevent buckling. In the next half cycle, the other brace is immediately ready to force the damper to slip in the opposite direction.

Figure 8. In-line Pall Friction Damper

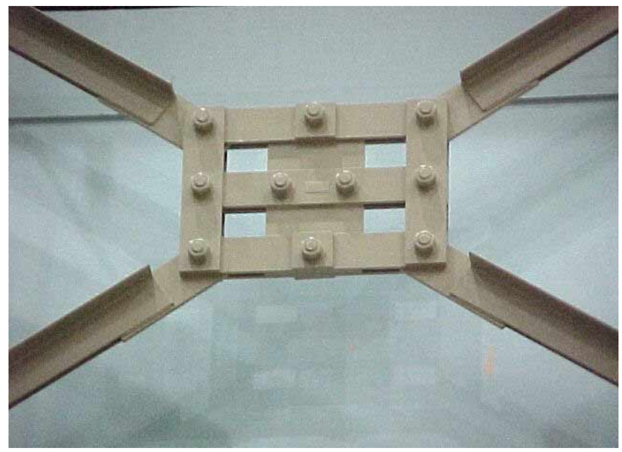

Figure 9. Cross Brace Pall Friction Damper

Where existing bracing members were not adequate, it was decided to replace them rather than strengthen. Replacing members rather than welding strengthening plates became more economical except in areas of difficult access where the installation became impossible.

The use of lead based paint on all early construction necessitated costly and time-consuming procedure to mitigate. Therefore it was desirable to develop designs that would minimize these procedures.

CONSTRUCTION

Retrofit of an existing facility is never easy even under ideal circumstances. The prime directive for construction for this project was not to impact aircraft assembly schedules. This made the coordination between construction and the Boeing production critical. The Boeing Construction Management Team handled the interface between construction and production personnel. Each temporary move was planned and coordinated well in advance so that there are no surprises. In one instance it was found that the existing gusset plates of about a dozen braces were placed 1.25 inch eccentrically. Instead of changing the gusset plates, the effect of eccentricity on the prototype friction damper was tested. As the effect of eccentricity on the friction damper performance was within acceptable tolerance, the gusset plates were not changed to save construction time.

Engineering staff worked closely with the field staff to minimize the impact of the construction on aircraft production and to simplify construction details. As the project progressed, many suggestions to simplify construction were incorporated into the engineering design.

CONCLUSION

The use of Pall friction dampers was very effective and practical for the seismic upgrade of this important structure. Since the forces on the structure were considerably reduced, the chosen scheme provided significant saving in both construction cost and time. Applying new technologies and developing construction details for a big project of this scale was very challenging for all the participants.

ACKNOWLEDGMENTS

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