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COST SAVINGS FROM IMPROVED SEISMIC EVALUATION, A CASE STUDY

J. Soulages[1], M. Sinclair[2], and M. Braund[3]

ABSTRACT

Over the last 6 years, we have undertaken the seismic evaluation of a 585,000 square-foot, six-story eccentrically braced frame office building built in the early 1990s. The goal of this effort was to develop a cost-effective and low-impact upgrade scheme utilizing ASCE-41 with the goal of life-safe performance. The progression of phases is described each using progressively more sophisticated analysis techniques from ASCE-41: linear dynamic, non-linear static and non-linear dynamic analysis. Construction costs and disruption are compared for four schemes developed during programming. The value of the more sophisticated evaluation techniques is dramatically demonstrated by comparing the cost and schedule savings achieved at each phase to the cost of the additional evaluation.

Introduction

As part of its ongoing effort to improve the life-safe performance of its buildings, Intel Corporation has undertaken a major effort to evaluate a number of buildings on its campuses in Santa Clara, California. This paper describes the results for one particular building and describes the 6-year effort to determine the most cost-effective and least disruptive strengthening scheme to bring the building up to life-safe performance as defined by ASCE 31 (ASCE, 2003) and 41 (ASCE, 2007). With each phase, additional investigation and more advanced analysis techniques were employed leading to dramatic reductions in cost and disruption to building occupants.

Building Description

The building investigated is a six-story steel frame structure of approximately 585,000 square feet that was constructed in 1992 using the 1988 Uniform Building Code. The first floor includes the building entrance and lobby, a museum, cafeteria, kitchen, mailroom, and a television studio. Upper floors are typically open office with a central spine of conference rooms and office support functions.

The building is L-shaped in plan with approximate overall dimensions of 372 feet by 308

[1] Senior Structural Engineer, Intel Corporation, Hillsboro, OR 97124
[2] Principal, Degenkolb Engineers, San Francisco, CA 94104
[3] Design Engineer, Degenkolb Engineers, San Diego, CA 92101

feet. Figure 1 is a typical building plan view. The building is founded on a concrete mat foundation that is thickened below braced frame columns. The typical floors and roof diaphragms consist of steel framing supporting metal deck and concrete fill floors. The building's lateral force-resisting system consists of well distributed steel eccentric braced frames (EBFs) that utilize wide flange beams and columns and hollow structural section braces. Most of the frames have the link located at the center span of the beam, but several have the link located adjacent to the column.

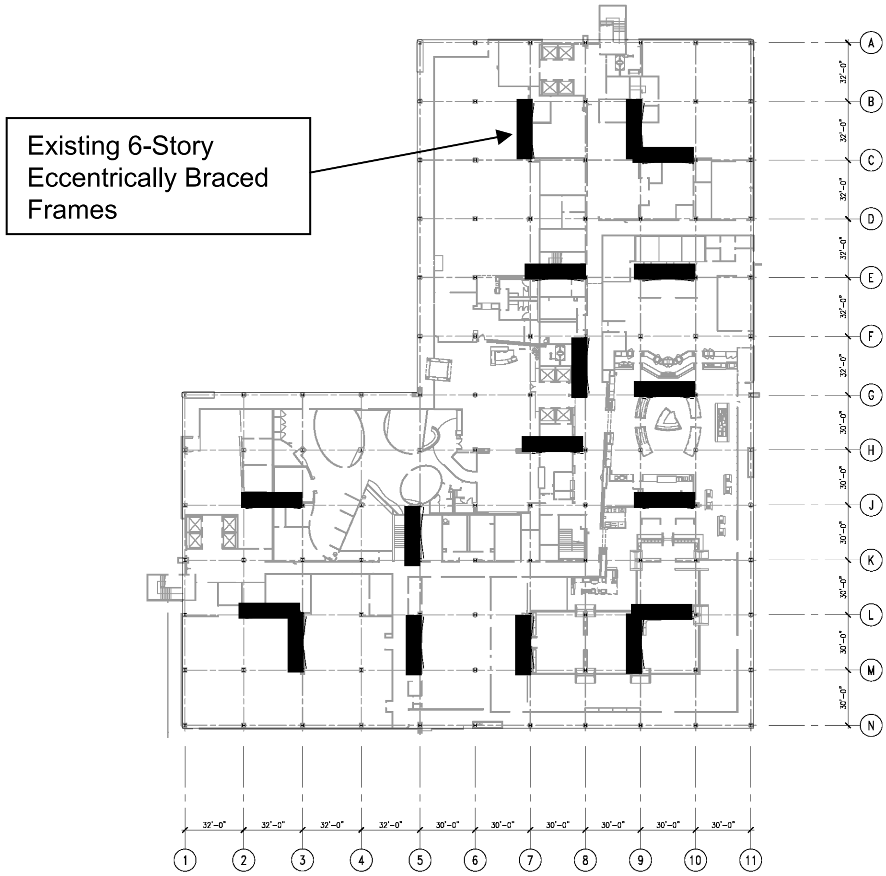

Figure 1. Typical plan view showing existing braced frame locations

Preliminary Seismic Evaluation

The first study undertaken of the building was a preliminary screening in August of 2003. This screening was roughly a one-week study that included the completion of a Tier 1 ACSE 31 checklist (with no analysis or follow up calculations), the compilation of a list of deficiencies, a very schematic upgrade approach and an order-of-magnitude upgrade cost based on the building type and square footage. The cost of the evaluation was roughly $2,500 USD.

The deficiencies identified for the building included: axial overstress in the braced frame columns due to overturning, diagonal bracing that did not comply with compact or slenderness criteria of the AISC Seismic Provisions, and braced-frame connections that could not develop the strength of the diagonals. Potential rehabilitation measures were suggested and included: strengthening or replacement of the braces and connections and adding cover plates to the columns of the braced frames. No foundation work was anticipated as the ASCE checklist required only "anchorage," with no strength requirement specified. The cost estimate based on typical costs was $6M to $12M USD for construction only with no indirect or disruption costs added. No construction schedule was estimated at that time. Because of the large uncertainty and the huge cost range projected, it was clear that additional investigation was necessary.

Detailed Seismic Evaluation

A detailed seismic evaluation of the building was undertaken in the summer of 2006 that included a site visit, full building linear dynamic analysis, a complete upgrade scheme including sketches, review of the scheme by architectural and MEP consultants and a cost estimate prepared by a local contractor. The cost of the detailed evaluation was roughly $100K USD.

One of the problems identified with the first analysis was that there was no specific ASCE 31 checklist for an EBF building – the checklist for a standard steel braced frame was used. This was a key issue because many of the beneficial plastic design provisions of eccentric braced frames were overlooked. In addition, the lack of a specific checklist missed a number of other deficiencies that could only be uncovered with a more thorough analysis.

It was found that although the building had a number of positive design features, including a good configuration of frames in plan, a large number of braced frames, no vertical irregularities, and excellent diaphragms and drags, the performance of the building was hampered by inefficient link beam member sizes. Only two link beam sizes were used throughout the six floors of the building and thus many were substantially over-designed. This greatly increased the demands in the beam beyond the link, the braced frame columns, the braced frame connections and most importantly, the connection of the braced frame to the foundation.

At the conclusion of the detailed study the proposed strengthening scheme included: strengthening all the brace members in the building by adding plates or filling the tubes with concrete, strengthening all the gusset plate braced frame connections by increasing the size of the fillet welds and strengthening all the EBF column base connections to provide additional tension capacity by extending the existing base plate and adding a large number of epoxy anchor bolts. The detailed cost estimate was $15.9M USD for hard construction only (including a 20% contingency). Estimated construction duration was 64 weeks. While the analysis and investigation were certainly more advanced than the previous preliminary study, the results were tending in the wrong direction – more costly, longer schedule and more disruption.

Detailed Seismic Evaluation Part II: Non-linear Pushover

One of the recommendations made at the end of the detailed study was to undertake a non-linear pushover analysis utilizing ASCE 41. The hope was that it would prove that the upper level links never yielded and thus the overall strengthening work needed in the superstructure could be reduced. Intel undertook the additional study in March of 2007. The cost of the additional study was roughly $50K USD.

After a complete non-linear analysis pushover model was developed and analyzed, it was discovered that significant flexibility in the foundation was a beneficial mechanism for limiting displacement demands on the superstructure and lowering the overall forces on the building. It was confirmed that many of the upper level links never yielded and that only certain braces in the first, second and fourth stories of the building were overstressed thus reducing the overall brace strengthening quantity by 70%. However, no change was able to be made to the base connection strengthening. Even with the lower forces, the demands on the connections were 2.5 to 4 times capacity at the mat foundation. The new detailed cost estimate was lowered to $10.2M USD for hard construction only (including a 20% contingency). Estimated construction duration was reduced to 56 weeks. Things were looking more promising but the cost was still high and the anticipated disruption at the first floor which is the most important floor of the building was considerable. It was decided to move into programming but to include in the effort additional tasks to continue to analyze the building and optimize the design.

Programming

Programming began in August of 2008 and included a number of tasks to continue to try and drive down the cost and disruption of the project:

- Destructive exploration of a number of areas of the building both to verify construction details (especially of the exterior curtain wall attachment) and to take steel coupon samples, concrete cores and samples of rebar in accordance with ASCE 41. It was hoped that by having more refined member properties, the forces generated in the analysis would be lower.
- Detailed disruption drawings and contingency estimates by an architect-led team with full MEP and contractor support including photographs and rendering simulation of all conditions.
- Upgrade of the analysis technique to include three-dimensional non-linear time history analysis in order to further refine the analysis results by modeling the rocking behavior of the EBF frames after pullout of the column base connections. This required the development of a suite of time histories for the site by the geotechnical engineer, AMEC Geomatrix.
- Investigation of several value engineering ideas generated in the previous study work including: further investigation of the strength of the long foundation anchor bolts and research into weakening the link beam with strategically placed holes.
- Most importantly, development of a wide variety of upgrade schemes such as adding new frames at the first floor, replacement of existing braces with viscous dampers, flipping brace directions at alternate floors from chevron to inverted chevron, adding outrigger frames to an upper floor, and upgrading only select base connections while allowing others to respond in rocking. Evaluation of all the schemes and selection of the final scheme was to be accomplished by a design charrette that included key representatives from the customer team.

The cost of the programming effort, including services by all the consultants, sample extraction and testing and complete documentation, was roughly $700K USD, the majority of which was related to sample extraction and site investigation.

Development of Alternate Strengthening Schemes

After creation of the new analysis model, a number of schemes were developed and analyzed using the new nonlinear model. By the time of the charrette, four primary schemes were presented and one additional scheme was discussed.

Scheme 1: Add Braced Frames at First Floor Perimeter

This scheme involved adding several new braced frames to the first floor perimeter (see Fig. 2). The primary concept was to transfer a percentage of the seismic load at the first floor to these new frames to reduce the flexural and, to a lesser extent, the tensile demand on the existing column base connections. The new frames also reduced the tendency of the structure to twist after column base connection pullout.

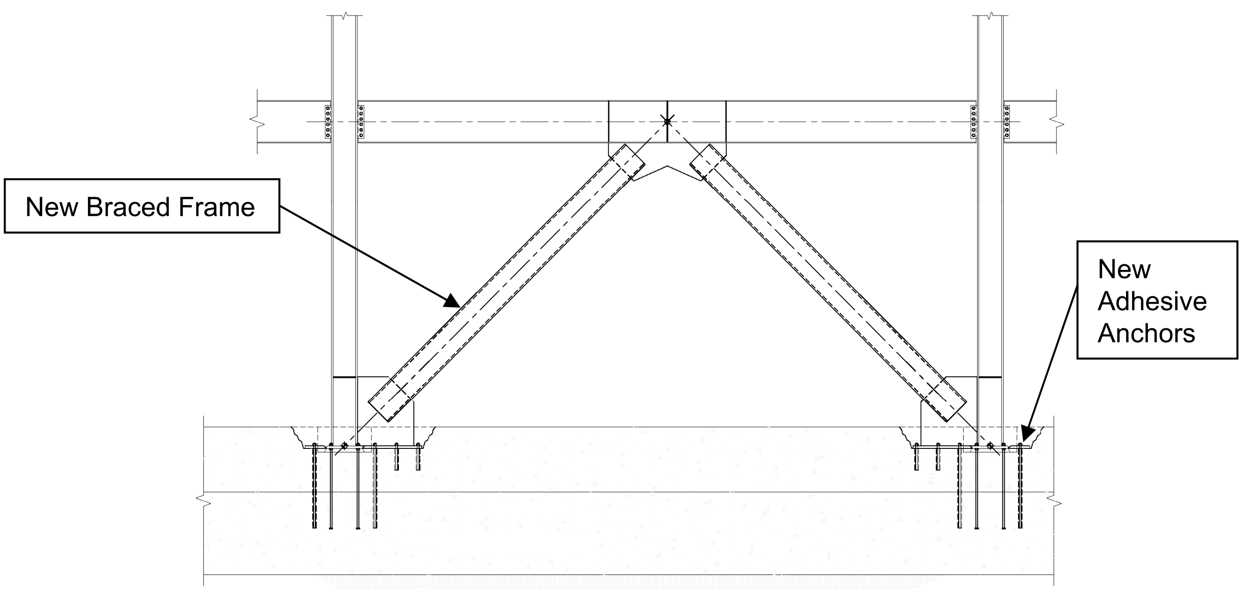

Figure 2. Typical elevation of new braced frame

Scheme 2: Add Outrigger Braced Frames at Sixth-Floor

This scheme involved adding outrigger-braced frames at the sixth floor, adjacent to the existing eccentrically braced frames (see Fig. 3). The concept was to reduce the uplift demand on the existing frame base connections by providing a wider overall frame. The frames also stiffen the EBFs and help force yielding of the upper level links. The sixth floor was suggested for the outrigger frames as it was much less disruptive and could be coordinated with a planned tenant improvement project involving that floor.

Although new braced frame bays would be required only at the 6th floor, it was discovered that the column splices below the new braced frames would require strengthening at the 3rd and 5th floors due to the large tension forces due to uplift.

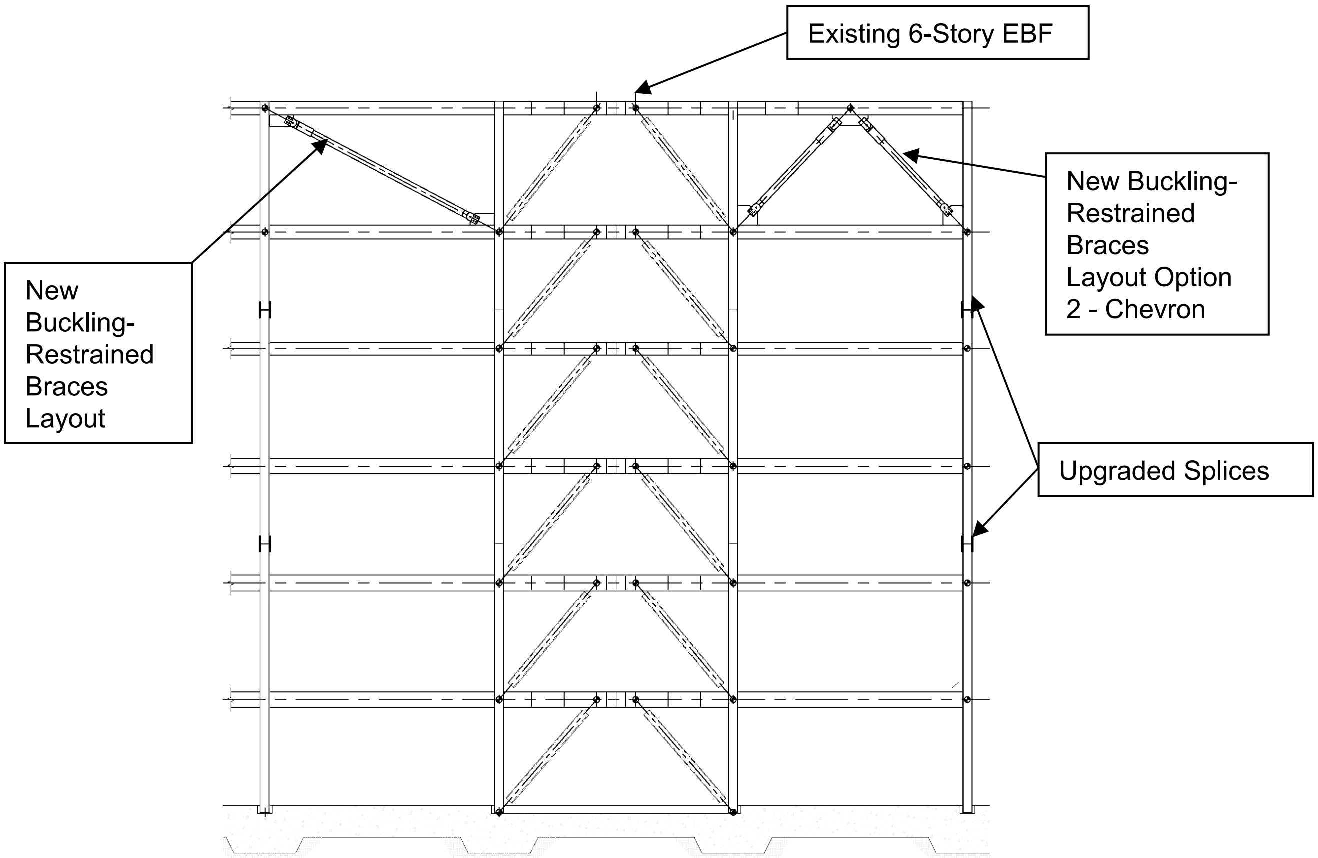

Figure 3. Typical elevation of outrigger braced frames at 6th floor

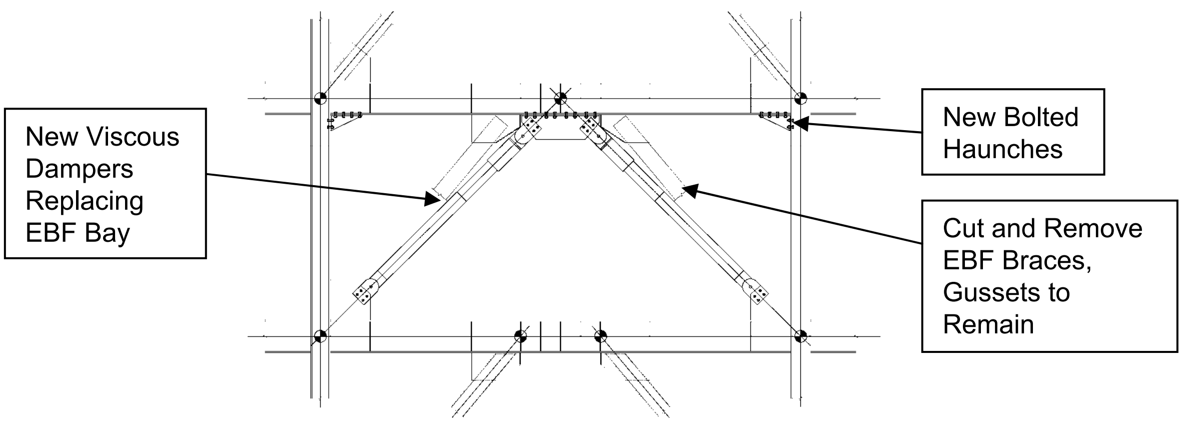

Figure 4. Typical elevation of EBF frame retrofit with viscous dampers

Scheme 3: Viscous Dampers

This scheme replaced select EBF bays (3 bays per direction) with viscous dampers on the 2nd through 6th floors (see Fig. 4). The concept was to reduce the demands on the column base connections by allowing the viscous dampers to dissipate energy and lengthen the building period. The potential drift increase due to stiffness loss was hoped to be compensated by the increased energy dissipation from the viscous dampers. The use of all-bolted connections was proposed to eliminate the need for welding within the still occupied building and reduce the amount of disruption due to welding protocol.

Scheme 4: Flip EBF Braces

This scheme flipped or converted EBF braces at alternate levels from a chevron to a "V" configuration. The concept was to eliminate alternate EBF links up the height of the building and reduce the demands on the column base connections. The increased building drifts would be accommodated by remedial work on the building curtain wall.

Base Strengthening Details

All of the four schemes developed required some strengthening at the base of the braced frame columns. Two types of column base strengthening were proposed: base anchors and base outrigger beams. The base anchor strengthening was proposed for all of the schemes. The more complex outrigger beams were required only in Scheme 1.

The proposed base anchors, as shown in Fig. 5, consisted of a number of long concrete cores filled with reinforcing bar and grouted solid arranged in a circular pattern around the existing column base plate. While they did not increase the initial strength of the foundation, they were found to provide residual tensile capacity at selected column base connections following pullout or cone failure of the existing anchor bolt group. The cost to install the base anchors was determined to be low relative to the improvement in performance.

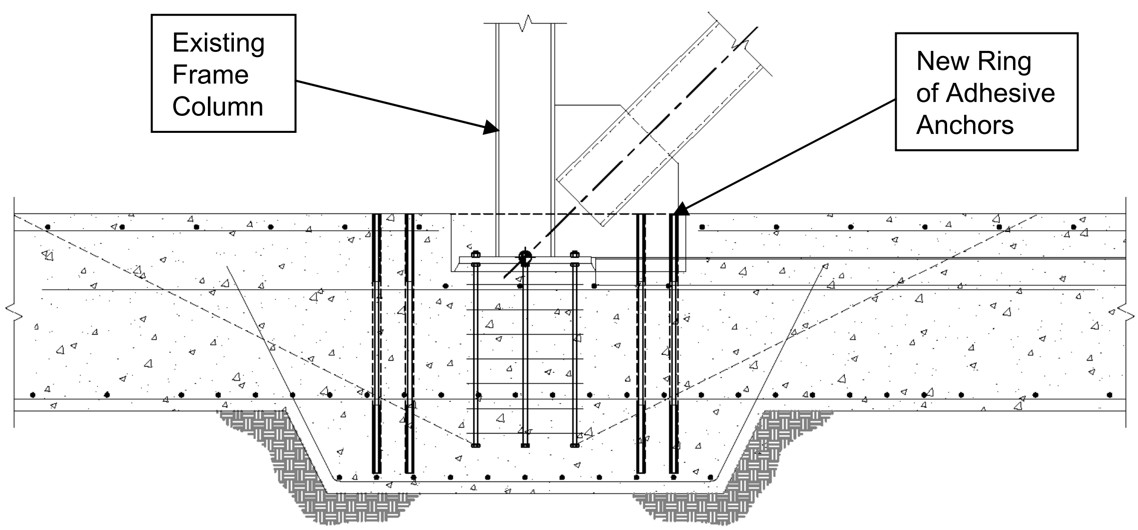

Figure 5. Base anchors at braced frame columns

The column base outrigger beams, as shown in Fig. 6, were proposed as a way to provide additional strength to the column base connection and mobilize the full capacity of the EBF above. The outrigger beams transfer a portion of the tensile and moment demand away from the column base to new anchors located on each side of the column. The most advantageous location for the column base strengthening was determined to be at the three pairs of intersecting braced frames located in each corner of the building.

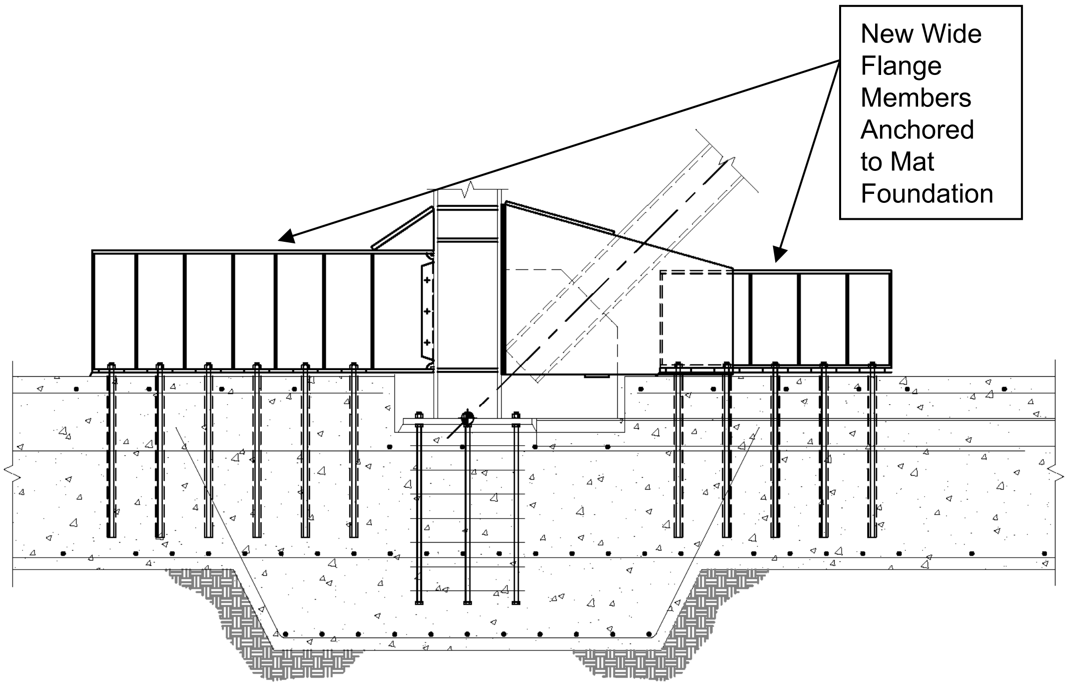

Figure 6. Outrigger beam section

Additional Strengthening Approach

There was one additional scheme investigated by the team: EBF link weakening. Since the existing EBF links were stronger than required and not behaving as deformation-controlled components, it was thought that the links could be weakened to limit the demand on the column base connection while still keeping lateral displacements to an acceptable value. Relatively early in the assessment process it was determined that the degree of weakening required was unfeasible and would result in poorly performing link and excessive building drifts. In addition, it had the added disadvantage of being perceived as "weakening" the existing system which would have implications for the ease of local jurisdiction review and approval.

Scheme Discussion and Selection

The design charrette was held in October of 2008. Participants examined and compared all of the schemes in terms of construction cost, phasing, operational disruption during construction, permanent disruption after construction, and long-term and short-term staff relocation. The contractor, Turner Construction, generated ROM construction costs for each scheme to aid the discussion. Because of a desire to not get bogged down with a detailed discussion of the estimates, it was decided to use relative costs normalized to the least costly scheme, see Table 1. In addition, the architect, IDC Architects, developed conceptual marked up photos to give an idea what each scheme would look like once completed.

Table 1. Scheme Selection Matrix

Scheme	Scheme Description	Operational Disruption*		Staff Relocation*		Normalized ROM Cost
		During Constr.	After Constr.	Short Term	Long Term	
1	1st floor braced frames, base anchors & outriggers	Mod.	Low	Low	Low	1.4
2	6th floor outrigger frames and base anchors	Low	Mod.	Low	Mod.	1.0
3	Viscous dampers and base anchors	High	Low	Mod.	Low	2.5
4	Flipped braces and base anchors	High	High	High	High	2.0

* Subjective Ranking: Low, Moderate, High

After a day of intense discussions, some clear choices were emerging. Scheme 4 was the first scheme eliminated. It was clear that by flipping many of the braced frames, the loss of total strength and stiffness would likely result in excessive lateral drifts that would end up damaging the exterior curtain wall. In addition, it would require work at most of the frame locations at up to three levels in the building which would be very disruptive and it would require the current floor layouts to be radically redesigned as the “V” configuration impacted the typical interior corridor.

The next scheme to be eliminated was Scheme 3. It was found to be considerably more expensive than the other schemes due to the large number of locations that required modification and the high cost of the viscous dampers and cast-bolted brackets.

The third and final scheme to be eliminated was Scheme 2. While the use of outrigger frames at the sixth floor was found to be the least expensive solution, it had the most total disruption including both temporary disruption during construction and more importantly permanent disruption for the life of the building.

In the final analysis, Scheme 1 with the frames on the first floor was selected as it had a number of advantages relative to the other schemes. First, half of the braced frames were able to be located beyond the exterior curtain wall where the building was set back. This allowed much less total disruption as all construction activities could occur completely outside the building. Several other braced frames at the interior of the building affected relatively low-impact areas. The use of braced frames at the first floor, as opposed to the sixth, was considered a more conventional approach and less likely to result in issues during regulatory review. And finally, the new exterior frames could be expressed and provide a visual indication that the building has been strengthened.

Final Analysis Refinement

After the design charrette was complete and the final scheme was selected, additional work was undertaken to further refine the scheme and reduce the scope. After complete analysis with the final suite of time history records, seismic forces were able to be reduced by an additional 10%

from the initial runs used to explore the various schemes. In addition, the A/E, contractor and Intel completed a final impact analysis. The following reductions were made: reduce the frames on the first floor by two frames, change the new braces to buckling restrained frames, replace all the outrigger beams with base anchors only, and remove two of the anchor base connections in the cafeteria area.

The final scheme was documented in a comprehensive programming report, published in January of 2009, which serves as the basis of the final design work. The final cost estimate was reduced to $3.5M USD for hard construction only (including a 20% contingency) which is a 60% reduction from the previous detailed study. The estimated construction duration was reduced to 24 weeks which is over a 50% reduction from the previous study.

Table 2. Summary of studies and costs

Study Date	Phase	Study Cost (USD)	Retrofit Cost (USD)	Schedule Estimate	ROI
Aug 2003	Preliminary Evaluation	$2.5K	$6 - $12M	N/A	-
June 2006	Detailed Evaluation	$100K	$15.9M	64 weeks	-
May 2007	Nonlinear pushover study	$50K	$10.2M	56 weeks	114x
Aug 2008 – Jan 2009	Programming with nonlinear time history analysis & alternate schemes	$700K	$3.5M	24 weeks	9.5x

As noted above, a significant portion of the programming study cost was associated with material testing and site investigation. Were these efforts of lesser scope, the ROI for the final phase would have been significant higher.

Conclusions

Every building is different and the results presented in this paper should not be considered typical. It is clear however that the return on investment of additional study utilizing advanced analysis techniques as well as alternate scheme exploration can be very high given the right building and a knowledgeable team. In this case, it produced a highly efficient design with a combination of low cost and more importantly low disruption to the existing building occupants both during and after construction. One has to realize however that this type of effort takes time and cannot be rushed. The key was to break the process into a number of small discrete steps with each step building upon the previous work and to allow enough time for each phase as multiple options are considered.

References

ASCE Standard 31-03, 2003, *Seismic Evaluation of Existing Buildings*, American Society of Civil Engineers, Reston, VA.

ASCE Standard 41-06, 2007, *Seismic Rehabilitation of Existing Buildings*, American Society of Civil Engineers, Reston, VA.