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OVER-RELIANCE ON COMPUTERS AND THE DECLINE OF GOOD ENGINEERING PRACTICE AND COMMON SENSE

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ABSTRACT

Computing power and speed have doubled every few years, computer software is increasingly sophisticated, and the costs of high-powered analysis have fallen dramatically. While these developments should all be positive for our industry, the authors have seen more and more evidence that computers also pose a significant danger to good engineering practice in both the technical and ethical arenas--particularly when it comes to earthquake engineering. Through five examples, the authors examine whether computers have contributed to a general improvement or a general decline of good engineering practice and common sense. Means of back-checking computer results are also presented, so that errors like these can be avoided in the future.

Introduction

Computers open the door to a universe of almost limitless possibility when it comes to engineering. Computing power and speed have doubled every few years, computer software is increasingly sophisticated, and the costs of high-powered analysis have fallen dramatically. While these developments should all be positive for our industry, the authors have seen more and more evidence that computers also pose a significant danger to good engineering practice in both the technical and ethical arenas -- particularly when it comes to earthquake engineering.

The authors were taught that a finite element analysis (or, for that matter, any computer analysis) should never be performed unless you already have a pretty good sense of the correct answer -- a sense developed from application of fundamental principles of structural mechanics and from experience. That essential paradigm is becoming lost amid the flash and glitter of late vintage computer programs. Through five examples, the authors examine whether computers have contributed to a general improvement or a general decline of good engineering practice and common sense. Means of back-checking computer results are also presented, so that errors like these can be avoided in the future.

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Case Studies

Case Study 1: Peer Review of a New Concrete Shear Wall Design

The authors recently participated in a peer review of two new five-story concrete shear wall structures. Each tower was rectangular, with post-tensioned flat slabs and a shear wall typically located at or near each exterior wall -- fairly simple structures to analyze. At the ground floor, some of the shear walls were discontinuous and were supported by columns or transfer beams. The project was approximately one-third built at the time we became involved; the contractor and several other engineers had expressed concern that the design was not constructible and that the lateral force resisting system detailing and design appeared deficient. The authors were retained by an owner's representative to evaluate the design and determine whether the claimed problems were real.

Problems with the Computer Model and Calculations

The authors were given access to the structural engineer's calculations and drawings, but were denied access to the engineer's electronic computer models, which the engineer claimed were proprietary. From the input and output files, it became immediately apparent that there were significant and serious problems with the computer model. Rather than computing the weight of the structures by hand, the engineer had allowed the computer to determine the weight and mass of the structures. While this can be a perfectly acceptable means of determining the weight, the engineer made no effort to adjust for the weight of nonstructural elements or to substantiate the weights generated by the computer. Not surprisingly, the computer failed to recognize that the structure had relatively thick nonstructural topping slabs, had interior gypsum board and concrete masonry unit (CMU) partitions, and had an exterior stucco cladding -- this error resulted in underestimation of the weight of the structure by approximately 30%.

The computer model had other significant problems as well. The governing building code, the 1997 Uniform Building Code (UBC), requires that all elements that are not part of the lateral force resisting system be designed to accommodate the maximum predicted inelastic deformation; in this case, the columns and flat slabs should have been designed to withstand the maximum predicted interstory drifts associated with the design earthquake. However, upon review of the calculations and computer output, it was apparent that during the deformation compatibility check, the designer had set the interstory drifts and moments equal to zero -- in effect negating the deformation compatibility check and resulting in the potential for punching shear failures during the design earthquake. When asked how his design complied with compatibility requirements, the designer stated that his computer model used "pin" connections between the slab and the column; therefore no moment could be developed. It should be noted that the engineer's design did not provide any means of ensuring that no moment would be developed between the columns and the slab; he merely assumed that none would develop.

Further compounding the obvious, the computer output stated that a number of cantilever shear walls needed boundary elements at the top half-story of the structure, and that no boundary elements were required for several stories below. Since boundary elements are used to resist the large compressive forces resulting from overturning, it was not clear why the top few feet of these walls would require boundary elements (where there is virtually no overturning) and why some of the floors below would not require boundary elements (where overturning moments should be increasing).

The designer had also made the common mistake of failing to properly define wall lengths. Where short cantilever walls met long foundation walls, the designer allowed the computer program to "smear" or average the horizontal shear stress over the entire length of the foundation wall. Since the computer program used by the engineer assumed that the diaphragm is infinitely rigid, and since the engineer had defined the long foundation wall as a single element, wall stresses fell dramatically as the infinitely rigid diaphragm distributed the loads from single-bay cantilever walls over the length of the foundation wall. For example, at one tall cantilever wall, the engineer specified horizontal shear reinforcing at the base of the wall to be #7 bars at 6-inches on center; yet, immediately below the (assumed) infinitely rigid second floor slab, the engineer reduced the reinforcing to #5 bars at 8-inches on center. In reality, a collector element would need to be designed in this location and localized stress concentrations (e.g. development of the shorter wall's boundary steel) in the wall would need to be dealt with at this area; the designer failed to address these issues. Other similar issues were also noted; the rigid diaphragm modeling assumption appeared to cause some walls to significantly unload in lower stories; thus horizontal shear reinforcing assigned by the computer program actually decreased in some of the walls on lower floors, despite the shear wall cross-sectional areas remaining unchanged.

Problems with the Detailing

The designer abjectly failed to provide ductile detailing. Since the designer took the computer's recommended reinforcement design -- based solely on force -- and copied it directly to his drawings, the design was never checked against the prescriptive detailing requirements of the code -- the very same requirements that attempt to ensure the necessary ductile behavior. Column ties, confinement reinforcing in columns supporting discontinuous walls, collectors, diaphragm boundary reinforcing, shear wall reinforcement at coupling beams, and required boundary element reinforcing were simply omitted because the engineer never checked the output of the computer to make sure that it complied with code requirements.

The design also failed to provide sufficient details to actually construct the structures. In this case, since the designer merely tried to replicate the computer output, the designer failed to provide many required details, and the design intent was not clear. Where 24-inch square columns at the first floor were connected with 14-inch diameter round columns on the second floor, no details were provided to show how the reinforcing steel for the two columns could actually be integrated. Since the reinforcing steel for the lower floor column was supposed to continue up into the upper floor column, the resultant steel reinforcing (28-#11 bars and 6-#7 bars) would have been more than 60% of the area of the confined core of the upper 14-inch diameter column -- definitely not a constructible detail. When the contractor asked what to do about the congestion, it is our understanding that the engineer told the contractor to simply cut the reinforcing from the lower column at the underside of the slab, precluding proper development of the reinforcing.

The engineer also failed to show 135° seismic hooks at the end of the transverse steel hoop reinforcement for the round columns; the engineer then approved the steel fabricator's shop drawings without catching the omission.

What Should Have Occurred

In this case, the engineer over-relied on his computer model and failed to perform even the most rudimentary checks regarding his design. The designer insisted that since he had done a "complicated analysis," it was simply not possible to check his design by hand. In fact, the designer had merely used a linear, dynamic procedure to design the structure. Since the structure was relatively simple and only five-stories tall, it was very easy to perform a weight take-off, determine code-prescribed lateral design forces, and distribute them to the two or three shear walls in each direction based on relative stiffness via an Excel spreadsheet. As it turned out, the design was seriously flawed, and the project was significantly delayed mid-construction while the engineer was required to fix his design.

Case Study 2: Analysis of a Collector

As part of a committee assignment, the authors reviewed a study on the effects of collectors in concrete slabs. A committee member had taken a four-story concrete structure with concrete shear walls and analyzed it; the analysis purported to show that properly designed collectors actually detracted from the performance of this very simple structure. The basis of the proof, as presented to the committee, was that the tension stress in the slab was two to three times as high when the collector was modeled compared to when the collectors were deleted. The investigator concluded that the presence of the collector causes a stress concentration near the wall, and that the stress distribution was more uniform where the collector was omitted. When asked for feedback, a number of committee members stated that the code was at fault for requiring collectors to be designed and that the structure was essentially responding to the presence of the collectors by overstressing them. Since the idea that a properly designed collector could actually detract from the overall performance of a simple structure seemed farfetched, and since the idea the structure was responding to the presence of the collectors by overstressing them was not only counterintuitive but also impossible, the authors reviewed the model in more detail.

Problems with the Computer Model

Upon review of the computer model, it became clear that the computer model was flawed. Firstly, the boundary conditions were inaccurate. Rather than model the entire four story structure, the engineer had taken a single floor and fixed the nodes at the shear walls. The infinite rigidity caused by the fixed nodes created an artificial spike in the stresses at the leading edges of the shear walls; stresses were exaggerated by the artificial restraints on the slab and were essentially meaningless, except to show relative differences between the models.

Proper interpretation of the results was critical to understanding what the output showed. Rather than explicitly model the large bundle of reinforcing steel that made up the collector to find out what loads the steel was taking, the engineer had increased the stiffness of the concrete elements at the collector to account for the increase in steel -- resulting in a transformed section. Since in an elastic model, the load attracted by the collector is directly proportional to the stiffness of the collector elements, the load in the collector elements necessarily went up, and so did the corresponding stresses. However, a direct graphical comparison of stresses between the model with the collector and the model without the collector is meaningless, since the load in the collector elements would have to be distributed between the steel and the concrete -- and this the computer model and the engineer did not do. Consequently, the model with the collector appeared to show higher stresses than the model without the collector -- which was interpreted as poorer performance. It is important to note that the computer model was actually modeling the increase in stiffness (and therefore the gross increase in stress) resulting from the transformed section correctly; but the graphical representation of the tension stresses was meaningless without back-transforming the stresses to account for all the steel in the collector.

What Should Have Occurred

The engineer analyzing the structure should have had a basic understanding of what answers he expected to get out of his finite element model. The conclusion that the addition of a collector -- whose sole purpose is to reduce localized shear transfer stresses, collect forces in a slab, and deliver them to a lateral force resisting element (such as a shear wall) via tension or compression -- could somehow cause a simple structure to experience higher and more localized stresses is ill-conceived. In this case, at best, the engineer could have proven that a collector was not needed. When the analysis results showed an apparent adverse effect from the collector, the engineer should have gone back and checked his boundary conditions and modeling assumptions. In this case, the addition of the collector may have been superfluous, but it certainly did not have an adverse effect on the overall performance of the slab or the structure as a whole.

Case Study 3: Analysis of Cracking and “Sagging” in a Concrete Slab

Years after the 1994 Northridge earthquake, an engineer was retained to evaluate cracking in an elevated concrete slab. The slab formed the second floor of a two-story parking garage; above the garage was a structural slab supporting a three-story wood-framed residential structure. The slabs were 10.5-inches thick and spanned as much as 27-feet. In places, cracks tended to radiate out from the columns; in other places cracking on the top of the slab was located in negative moment regions. In a few bays of the structure, the slabs appeared to “sag” approximately 7 inches from the perimeter of the structure to the middle of the drive aisle, based on the results of a manometer survey. The engineer responsible for evaluating the structure constructed a finite element model of the garage, identified nearby ground motion recording instruments and records, subjected the model to the vertical component of the Northridge earthquake, and characterized the structural damage resulting from the earthquake as “moderate to severe”. In his report, the engineer plotted the various vibrational mode shapes of the slab and commented that the largest “dips” in the mode shapes corresponded to the bays in which the largest deflections were measured. The engineer recommended removal and replacement of portions of the slab, application of fiber-reinforcing to the undersides of the slabs, and epoxy-injection of the cracks. Inexplicably, the engineer failed to recommend any temporary shoring or temporary restrictions on the fully occupied structure, despite the claimed “moderate to severe” structural damage.

Problems with the Computer Model

In this case, the engineer appeared to be relying largely on his computer model in the determination that the 1994 Northridge earthquake had caused the “moderate to severe” structural damage. However, the computer model results were suspect. The engineer claimed that since the predicted mode shapes were similar in shape to the “sag” observed in the slab, the earthquake must have caused the damage but the engineer did not provide any assessment of bending stresses or deflections resulting from the earthquake. Since the engineer’s computer model was not provided, the authors assembled a finite element model that replicated the modal shapes and periods of the engineer’s model. According to the engineer, the peak vertical ground motion associated with the Northridge earthquake at the location of the building was only 0.1g (10% of gravity) -- a very small percentage. The engineer claimed that vertical spectral accelerations on the order of 0.4g (40% of gravity) may have occurred. As it turns out, the modal periods of the structure were very short (0.18 seconds or less), and one wonders why any engineer familiar with earthquake engineering would believe that a slab with a fundamental period of 0.18 seconds, exposed to a spectral acceleration of 0.28g, could somehow experience 7-inches of “sag” when a spectral displacement of only 0.09 inches would be predicted.

Furthermore, after adjusting the spectral accelerations by the effective mass of each mode, the equivalent vertical load ranged from 25 psf to 40 psf, depending on which bay of the elevated parking slab was analyzed. We note that the equivalent vertical load resulting from the earthquake is substantially below the design live load of 50 psf for the garage and is also likely substantially below any construction loads that may have been applied to the structure, such as stacking/storing of construction materials and/or shoring of the floor above. We also note that estimated design load of 50 psf may not necessarily be conservative, given the public’s increasing penchant for larger and heavier sport utility vehicles (SUVs), and that equivalent uniform loads resulting from SUVs exceed 50 psf for structural members supporting tributary areas of 350 sq. ft. and less (Malik 2002).

When the unfactored stresses and deflections of the slab due to the applied loads were determined, the results were not supportive of the thesis that the Northridge earthquake had caused the “sag”. Table 1 shows the results of the analyses.

Table 1. Results of computer analysis on the two-way slab.

Loading	Maximum Bending Stress (psi)	Deflection (in)
Dead Load	270	1.2
Live Load	100	0.18
Vertical Earthquake Load	90	0.13

Note that bending stresses were computed using the gross slab area for means of comparison, and that the dead load deflection includes an estimate of the effect of long-term creep on these relatively long-span, two-way slabs. Since the Northridge earthquake occurred in the early morning hours, and since the claimed “sagging” occurred in the drive aisles, the live load and the vertical earthquake load would not be expected to be additive. In this case, it appears that by no stretch of the imagination could the observed cracking and “sag” have been caused by the Northridge earthquake. Vertical earthquake loads are clearly eclipsed by both the design live loads and by the substantially larger dead loads.

Conflicts with Reality

In this case, the claimed damage was directly rebutted by the objective physical evidence -- that of paint in the cracks. Like most garages, the floor had paint stripes to demarcate the parking stalls and traffic patterns; these stripes were installed soon after construction -- years before the 1994 Northridge earthquake. The striping had never been repainted, so if paint were to be found in the cracks, reason would indicate that the cracks must have been present when the stripes were first painted, soon after construction. In the case of this garage, every single crack that the authors examined that was crossed by a painted line had paint in the crack. Consequently, it was clear that the cracks were not from the Northridge earthquake, but rather stemmed from drying shrinkage and flexural cracking that occurred before the garage was even striped.

We also noted that the water-level survey conducted by the engineer -- if the engineer’s conclusions were to be believed -- appeared to show that a number of columns had moved downwards by as much as 3 to 4 inches. Again, the objective physical evidence would indicate that these columns did not move downwards relative to the other columns, since no corroborative evidence of distress was found on the uppermost slab or in the wood-framed residential units located above and supported on the concrete structure.

The engineer’s computer analysis notwithstanding, if the earthquake did not cause the “sag” in the elevated slab, then what did? In this case, one merely had to review the original architectural and structural drawings to determine the cause of the “sag”. The architectural drawings clearly required the top surface of the elevated concrete slab to vary so that the slab sloped to a number of drains -- including drains located in the middle of the drive-aisle with the “sag”. As-designed, the difference in elevation between the perimeter walls and the tops of the drains was as much as 7.4 inches -- more than enough to explain maximum “sags” of 7 inches and to explain the apparent 3- to 4-inch downward “drop” of some of the columns. Furthermore, the original structural drawings even contained instructions to the contractor to “warp the slab” to provide the profile required by the architectural drawings. When we observed the slab, it was clear that the slab sloped to drains provided in the middle of the drive-aisle. Based on all available evidence, the slab was constructed in general conformance with the original structural and architectural drawings; the engineer evaluating the structure appears to have mistakenly identified “slope-to-drain” as earthquake damage.

We note that the fact that the engineer never recommended shoring, temporary limitations on use, or any other reductions in load or occupancy as a result of the “moderate to severe” structural damage further belied the claimed damage.

What Should Have Occurred

By relying on a computer analysis, but ignoring the most important output (stresses and deflections) and ignoring the objective physical evidence, the engineer claimed moderate to severe structural damage had occurred as a result of a long-ago earthquake -- “damage” that was clearly original “slope-to-drain” specified by the original architectural and structural drawings. Not only did the engineer attempt to use technical jargon and the results of an incomplete computer analysis to attempt to drive his client into a very expensive and disruptive repair, but having alleged “moderate to severe structural damage”, the engineer also created a significant disclosure issue for his client when the client wants to sell the property. We note that had the engineer examined the stresses and deflections in his computer model or plotted the results of his analysis using the Acceleration-Displacement Response Spectrum (ADRS) format (Mahaney *et al*. 1993), it would have been immediately apparent that the minor earthquake demands could not account for the “sags” in the slab.

Case Study 4: Analysis of a Dual System

This case study deals with a concrete shear wall and steel moment frame dual system structure designed in the early 1960s. In this building, the steel moment frame was a robust and complete space frame, designed to support all of the dead and live loads on the structure and designed to have the strength to resist 25% of the lateral design loads at the time the structure was designed. However, the connections between beams and columns used high strength bolts, rather than welded connections used today. During litigation unrelated to the original construction, the question arose as to whether the original design qualified as a dual system as defined in more modern codes. One of the engineering firms that tackled this question used a common and fairly simple computer program to perform a design check on the existing steel frame; their analysis applied dead and live and lateral loads to the bare steel frame and then checked the result to determine whether or not the steel members were in compliance with modern code. In this case, the engineer relied on "canned" post-processing software to take the results from the computer program and compute the demand-to-capacity ratios. Numerous beams were shown to have demand-to-capacity ratios far exceeding 1.0; with some as large as 3 or 4, and the engineer concluded that the existing steel frame lacked the required strength to resist vertical and lateral loads specified by modern codes.

Conflicts with Reality

In this case, a closer inspection of the demand-to-capacity ratios would have indicated significant problems with the analysis. The engineering firm had analyzed the steel frame for a number of load cases that included full dead load and representative live load for all cases, combined with various levels of lateral load. Interestingly, the demand-to-capacity ratios almost remained unchanged whether the lateral load applied was 25% of that required by modern code or 100% of that required by modern code. In effect, the post-processing program was saying that whether or not lateral load was applied, the steel moment frame was substantially overstressed (demand-to-capacity ratios of 3 to 4) from just dead and live load. Since overstresses of this magnitude would generally be expected to result in significant deformation, distress, and possibly collapse, but since the structure had clearly performed well for over three decades without experiencing unacceptable behavior of the steel moment frame, the analyses were quite clearly incorrect.

What Should Have Occurred

The analysis conducted by the engineering firm was performed a number of years after they had already pronounced the steel frame to be sufficiently inadequate to qualify as the back-up moment frame for a dual system. The analysis also appeared to have been conducted in haste. Had the engineering firm checked the results before publishing them, even a cursory look at the post-processor output would have revealed that the post-processor had completely neglected the lateral support provided by the cast-in-place concrete slabs into which the upper flanges of the beams and girders were embedded, and ignored the thick cast-in-place and reinforced concrete fireproofing that completely encased most of the beams. Consequently, the post-processor improperly assumed that lateral-torsional buckling controlled the behavior of the beams and girders, resulting in apparent massive overstresses of the frame; the overall effect of the code-mandated lateral load on the frame was small and not the cause of the apparent overstresses.

Failure to account for the actual configuration of the structural systems and consequent beneficial restraint resulted in dramatically inflated demand-to-capacity ratios and nonsensical results. Proper checking and modeling of boundary conditions and assumptions would have precluded this outcome.

Case Study 5: Analysis of Cracking in a Concrete and Concrete Masonry Unit Structure

Years after the 1994 Northridge earthquake, an engineer was retained to evaluate cracking in a seven-story concrete and concrete masonry unit (CMU) structure and determine whether the damage was caused by the Northridge earthquake. The engineer performed a number of site visits to observe the structure, created an apparently detailed three-dimensional finite element model of the structure, analyzed the data, and concluded that based on all available evidence, the structure had experienced "moderate to severe structural damage" as a result of the earthquake. The engineer further claimed that comparison of the computer model output with observed cracking had yielded "excellent correlation". Similar to Case Study #3, the engineer failed to recommend any shoring or temporary restrictions on use for the fully and continuously occupied residential structure, despite the claimed "moderate to severe" structural damage.

Conflicts with Reality

In this case, the primary and most severe damage claimed by the engineer was cracking of some transfer girders that support discontinuous CMU shear walls. The engineer claimed that the earthquake had exceeded the yield capacity of the girders in bending by "as much as 12%", resulting in claimed "heart stopping" severe damage to the lateral force resisting system. When the authors reviewed the calculations for the beams, we discovered that the engineer evaluating the beams had used the nominal minimum yield strength to determine the bending yield strength. When the expected or median strength of the steel was used -- since this provides the best estimate of the actual yield strength of the beams -- the beams would not have been expected to yield. Furthermore, the engineer either deliberately or accidentally pinned or released the end connections of the cast-in-place transfer girders to the cast-in-place concrete columns. The failure to provide proper boundary conditions further overestimated the demands on the transfer girders. If the proper expected steel strength had been modeled and if the proper boundary conditions had been modeled, the only conclusion that could be drawn from the computer analysis is that the earthquake demands did not even come close to exceeding yield -- much less ultimate capacity -- of the transfer beams.

In addition, if the output from the engineer's computer model was correct, very large midspan negative moments would have occurred in the same beam as the discontinuous wall tried to uplift. The calculated negative moments were approximately 90% greater than the maximum ultimate negative capacity of the girder. Consequently, based on the engineer's model, one would have expected to see tremendous negative moment cracking in the transfer girders. Instead, there was a complete absence of negative moment cracking, indicating either that the engineer's model or his estimate of the severity of the ground input was substantially in error.

Since the transfer beam was restrained at both ends by cast-in-place concrete and reinforced masonry, the beam was restrained from shrinking after the concrete was placed, resulting in shrinkage cracking. As it turns out, the transfer beams had a significant discontinuity in reinforcing away from the areas of maximum bending moment -- and this discontinuity likely resulted in concentration of drying shrinkage restraint cracking at this location. Thus, the "heart-stopping" "severe" damage to the transfer beams was likely shrinkage cracking combined with some flexural cracking that had been present since the structure was built, long before the Northridge earthquake.

Furthermore, throughout the structure, the engineer generally failed to document the condition of the cracks -- i.e. whether paint, carpet glue, prior repairs, or other potentially age-revealing conditions were present within or around the cracks. In a number of locations, it was clear that repairs had been made to the area at some time in the past; however, the engineer made no mention of when the repairs had been performed -- information that could be critical to discovering when the damage actually occurred -- and in some locations, even failed to mention that prior repairs were present.

Claims of Excellent Correlation

Given that a claim of "excellent correlation" had been made, the authors undertook a study to assess whether any such correlation existed. In addition to maps and photographs of cracking, the engineer had included in his report stress plots of the various floor slabs and wall elevations that showed the claimed stresses from the Northridge earthquake. It was therefore possible to evaluate whether or not each instance of photographed cracking was located in areas that had, according to the computer analysis, experienced significant stresses as a result of the earthquake -- and thence <u>might</u> have been caused by the earthquake -- or were located in areas of low stress -- and were therefore very unlikely to have been caused by the earthquake (assuming that one even believes such a flawed analysis). It is important to note that "correlation" of a crack with an area of high stress is only correlated if the crack had not formed prior to the considered earthquake by some other cause (such as restraint to drying shrinkage, thermal movement, differential settlement, dead or live loads, prior earthquakes, etc.) -- making correlation very difficult to prove if you have not considered and been able to rule out other possible causes of cracking or movement in the structure prior to the earthquake under consideration.

In this case, 41 instances of cracking were presented in the engineer's report with sufficient information to determine where the cracks occurred. Of the 41 instances of cracking, 34 were either located in areas of low stress or were located in areas not even modeled by the engineer. Of the 41 instances of cracking, only four were located in areas of even moderate stress, and only three cracks were located in areas of high stress and could <u>potentially</u> have been caused by the earthquake -- but only if the cracks had not been caused by something else prior to the earthquake.

Given that there actually appeared to be little if any correlation between the computer model and the physical location of the cracking (for more than 90% of the damage, there was zero correlation), it was not possible to conclude that the engineer's claims of "excellent correlation" were valid. The owner may have been more likely to arrive at a better "correlation" by using a random number generator or flipping a coin.

What Should Have Occurred

In this case, the engineer appeared to have embarked on a course to determine that a specific earthquake caused the damage, whether or not the objective physical evidence and the computer analysis actually provided sufficient evidence to make such a claim. While visually pleasing, the engineer's computer model contained artificial and incorrect boundary conditions and modeling assumptions that did not represent the actual construction of the building. The artificial boundary conditions and improper modeling gave answers that supported the engineer's thesis, but when corrected, proved his thesis to be incorrect. Claims of "excellent correlation" were made, but when compared with objective physical evidence (i.e. where the cracks were actually located), correlation was absent. We note that in checking one's computer analysis, it is important to check the results against physical evidence and resolve any discrepancies. Similar to Case Study #3, this engineer made outrageous claims of "moderate to severe" structural damage and attempted to justify the claim by providing computer analysis results that showed the exact opposite. Also similar to Case Study #3, this engineer did not recommend any shoring, temporary limitations on use, or any other reductions in load or occupancy as a result of the "moderate to severe" structural damage.

Conclusions

In each of the case studies presented above, significant design and analysis decisions were based on the results of a "complicated" or visually pleasing computer analysis. However, errors in element meshing, errors in assigning boundary conditions, and errors in basic assumptions corrupted the results and gave incorrect and misleading results. Failure to properly interpret the results from the computer analyses also resulted in misdiagnoses. In an age where it is very easy to make an aesthetically pleasing finite element model, performing a "reality check" has never been more important. When the results of a computer analysis conflict with the objective physical evidence, the most likely explanation is that the computer analysis is incorrect. Simple hand calculations and actually observing the physical condition of the element or structure being analyzed can provide invaluable information that simply cannot be provided by a finite element model. No matter how fancy and powerful the software, results must be back-checked and common sense and ethics must be maintained; as a popular superhero once stated, "With great power, comes great responsibility".

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