How Not to Design for Moderate Seismic Regions

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ABSTRACT

Based on a collection of building peer reviews in moderate seismic regions, this paper discusses the manifold ways that both the spirit and the letter of the building codes and seismic design provisions can be misinterpreted, resulting in designs that are both unsafe and uneconomical. In light of these examples, the paper discusses the challenge of achieving in the building code an appropriate balance between simplicity, sophistication, transparency and prescription. Specific attention is given to design approach, architectural coordination, constructability and cost.

INTRODUCTION

This paper presents a number of examples of actual designs which were reviewed by the authors' firm, resulting in flaws found in the seismic lateral load resisting system. These examples are not presented to portray any superiority in the authors' practice, since some of the errors identified could easily occur within our own office. The sole purpose of these examples is to give designers, detailers, researchers and Code writers something to reflect on while they move forward in their work.

In recent years, Code development has mirrored the fast-paced changes occurring in other aspects of the design and construction community with software development, Building Information Modeling (BIM), safety consciousness on construction sites, and full-scale testing at a number of Universities to gather real performance data. An entire generation now exists that grew up with no other reference for analysis, design and construction than one based on using the computer to perform their tasks and communicate with others.

How can it be then, with the most sophisticated analysis and design tools, the ability to view a complete 3-D model of the building, more advanced research than ever before conducted, faster incorporation of research into Code writing and the ability to communicate with a far broader network than ever before, that we see more serious design flaws than ever before? The answer to this question lies, in part, in the asking. Relying on computers to generate design solutions and coordination and to keep up with research and Code development is flawed. Design of a building's structural systems requires careful consideration of the Architect's and Owner's goals and then implementation of Code requirements.

A review of the examples presented herein will result in categorizing the errors into one or more of the following categories:

1. The structural engineer spent too little time in the early stages of the design to coordinate an appropriately efficient lateral load system, and thus was left to use an inefficient or unsafe solution.
2. The structural engineer specified a lateral load system which required special seismic detailing requirements, none of which had been examined during design. When the low

bidding fabricator began to try to detail these requirements, costs escalated, schedules were threatened, and the Architect, General Contractor and Owner had no idea why.

3. The structural engineer relied on computer software to both analyze and design members of the lateral load system without appropriate checking of the loading impact, boundary conditions, diaphragm modeling and results. In some cases, this comes to light in the shop drawing stage and changes ensue, with costs escalating, schedules threatened, and the Architect, General Contractor and Owner dissatisfied.

Example No. 1

One-story steel-framed platform supporting 4-story wood-framed residential structure somewhere in the Midwest. The design team was from another state. The architect had recently had a bad experience with the structural engineer, and therefore requested an independent structural engineering peer review. What started out as a seemingly simple assignment escalated into a number of unfortunate conditions. The project was to be permitted under IBC 2000 with certain State Amendments which applied to snow, wind and frost, but did not impact seismic design. The building was situated on the site of an old ash fill. The Geotechnical Report classified the site as Site Class D, even though the blow counts in their borings showed N (blow count) values less than 15 per IBC Table 1615.11, which would suggest Site Class E. If the Site Class was E, the Seismic Design Category would have changed from B to C. Subsequent inquiries with the Geotechnical Engineer resulted in recommendations for ground improvement to support the Site Class D classification.

The General Notes sheet for the structural drawings indicated the lateral load system for the one-story steel frame was an ordinary concentric braced frame with R = 5.0. The S_{as} and S_{a1} were correctly defined. A review of the braced frames showed HSS member sizes which did not meet the prescriptive KL/r requirements of the AISC 1997 seismic provisions which were applicable under IBC 2000. The braced frame elevations did not provide connection design forces to design for. The General Notes sheet called for ASTM A307 anchor bolts. ASTM A307 anchor bolts are not recommended for cyclic loadings, such as may be expected from seismic or wind loads. The seismic tension force in the braced frame column, using the amplified load according to AISC Seismic provisions for R = 5 design, was approximately 10 times the capacity of the A307 bolts.

The floor beams, which were part of the braced frames, were not labeled in the braced frame elevations; instead the sizes were labeled on the floor plans. The beams shown on plan, which were part of the composite frame gravity system transferring the gravity and lateral load of 4 levels of wood-framed framing, did not have any shear connectors. In some instances, the floor beam in the braced frame was as small as a W10x12, even though the bracing member was an HSS8x8x1/2.

It was becoming obvious that a computer-generated design had been prepared which ignored AISC prescriptive requirements, was not checked for compatibility with standard general notes, and ignored rigid diaphragm assumptions of the computer software being used, and the Peer Review comments prepared by LeMessurier Consultants to address these issues were not understood by the architect, who was the liaison to by the structural engineer-of-record.

Meanwhile, the steel fabricator was preparing shop drawings under pressure to meet the construction schedule. The detailer submitted an RFI requesting the design forces in the braces, and the response by the SER was to develop the full capacity of the member in accordance with

the governing AISC seismic provisions and the listing of R = 5 on the General Notes sheet. The steel fabricator and their knowledgeable detailer quickly replied that seismic detailing was not required in their jurisdiction.

At this stage we attended a meeting with all parties and tried to explain that an R = 3 design was legal in the State where the project existed, and R = 5 design did require seismic detailing. While we were confident in our interpretation of the governing law, we did nothing to inspire confidence in the Architect, Owner or General Contractor regarding the design of the structure, nor what would ensue between the structural engineer-of-record and the steel fabricator as it related to cost and schedule.

Eventually, the structural engineer issued connection design forces for the braces, and these were appropriately calculated in accordance with the 1997 AISC seismic design provisions. The resulting connection forces were only slightly higher than what the fabricator would have expected from an R=3 design, since the R=5 design forces, amplified by $\Omega = 2$, were equivalent to R = 2.5. However the confusion and delays did not leave a good taste in anyone's mouth.

Example No. 2

A seven-story building in the Midwest has perimeter moment frames in the long direction and braced frames in the short direction. Spandrel bay sizes vary from 15' to 35'. The General Notes sheet lists the seismic lateral load system as ordinary moment frame using R = 3.5 with $\Omega = 3.0$.

The typical detail sheet showed a beam-to-column moment connection reflecting AISC seismic provisions including full-penetration welds of beam flange-to-column flange, removal of bottom flange backing bar and application of reinforcing fillet weld, continuity plates in the column to match the thickness of the beam flange plus 1/4", and web doubler plates illustrated on the column web noted "web doubler plate as required".

No additional information concerning the design of the moment connectors, other than the gravity reaction, was listed on the plans.

Typical column sizes were W14s, and spandrel beam sizes were generally W24s. The building had tall floor-to-floor heights with 15'-0" first to second, and 14'-0" typically above.

When the successful low-bidding steel fabricator submitted his bid to the general contractor, he included a qualification which excluded the notation on the Typical Detail Sheet "web doubler plate as required". The general contractor assumed this was a standard exclusion and did not share any information with the engineer concerning same.

During the early stages of the detailing, the fabricator submitted Typical Details and calculations for the standard connections, including the moment connections. The engineer rejected the submittal and requested calculations for the moment connections which conformed to the Chapter 11 AISC Seismic Provisions for Ordinary Moment Frames. The detailer's engineer designed the connections, and the resulting web doubler plates were in excess of 2". In addition, the oversized thickness of continuity plate could not be developed in welding to the web of the W14 without using a full penetration weld.

The resulting connections were to be very costly, and in some cases impractical to fabricate. At this stage, the fabricator was submitting a large Change Order for the work and suggesting a schedule delay as well for the added work. The general contractor and Owner had no idea what was going on and just wanted a safe building on time and on budget.

Fortunately, a meeting was held between the fabricator's connection designer and the engineer-of-record, where the connection designer cited many examples of ordinary moment

frame building designs where the engineer-of-record had supplied the design forces for the beam-to-column connection, and did not rely solely on the prescriptive requirements contained in Chapter 11.

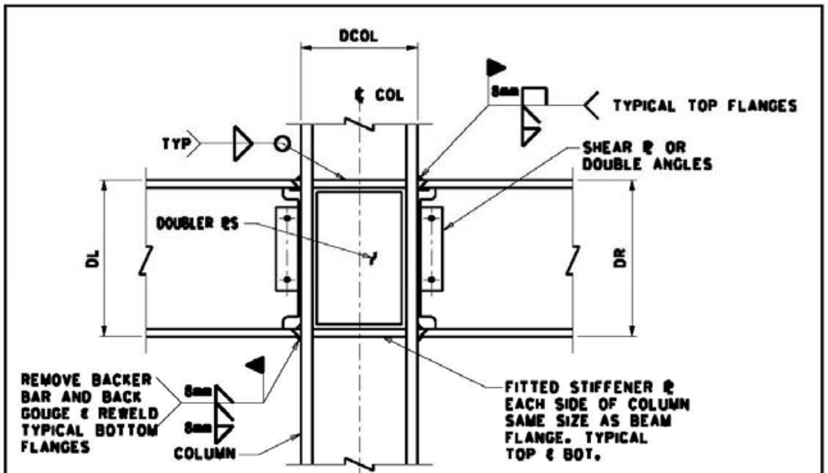
The engineer-of-record reviewed the original lateral load analysis and realized that most member designs were controlled by stiffness of the moment frame under wind loading. In fact, the member forces, even when amplified, were well below the elastic limit. In fact, this will almost always be the case in the usual moment frame design, unless the column spacing and floor-to-floor height are equal.

By now the engineer-of-record was realizing that the Chapter 11 seismic provisions allow for ordinary moment frames to be designed for "the maximum moment that can be developed by the system", and the wording in Section 11.2.a. commentary:

> *"It is reasonable to limit the requirements to the maximum moment that can be developed by the system, because the size of the beam or girder may have been determined to meet demands greater than the seismic demands. Factors that may limit the maximum moment that can be developed in the beam include the following:*
> *(1) The strength of the columns;*
> *(2) The strength of the foundations to resist uplift;*
> *(3) The limiting earthquake force determined using R = 1. "*

In what turned out to be a fairly simple exercise, the engineer-of-record returned to his original computer design and summary spread sheets which already had all the data for the appropriate load combinations. This information was shared with the connection designer, and the connections were designed and re-detailed accordingly. See Figure 1 for example analysis. Some web doubler plates were required, but the number and thicknesses were greatly reduced such that the fabricator submitted only a modest Change Order and no schedule delay, which the Owner and General Contractor accepted.

The Engineer-of-record decided that it would have been far easier to have listed the connection forces at the time the drawings were prepared or to perform the actual analysis for the web doubler plates at the time the drawings were prepared, when all the information and engineering staff were available, than to wait for the shop drawing stage when retrieving the information is more difficult to do.



ORDINARY MOMENT FRAME CONNECTION DETAIL
R = 3.5

COLUMN
W14x120
D_{COL} = 14.5 in
t_w = 0.59 in

LEFT BEAM
W24x162
D_L = 25 in
Z_{xL} = 468 in^3
M_{EQL} = 94.2 ft-kip (LRFD)

RIGHT BEAM
W24x162
D_R = 25 in
Z_{xR} = 468 in^3
M_{EQR} = 94.2 ft-kip (LRFD)

Panel zone shear per AISC 341-05

11.2A - Based on plastic moment capacity of the beam

$M_u = 1.1*R_y*M_p$ where R_y = ratio of expected to min yield stress

C11.2A - Based on limiting seismic force determined using R=1.0

$M_u=R*M_{EQ}$ where M_{EQ} = Governing LRFD seismic moment
R = Ductility factor for the system.
(multiplied to back out the ductility factor)

Shear demand on panel zone

$V_u = M_{uL}/D_L+M_{uR}/D_R$

Panel zone web shear capacity

$\Phi V_n = \Phi*0.6*F_y*t_w*D_{COL}$ where Φ = 0.90

Comparison Calculation:

Column web capacity:
ΦVn=0.9*0.6*50ksi*0.59in*14.5in=231 kip

Req'd doubler plate thickness based on plasic moment capacity:
M_u = 1.1*1.1*50ksi*468in^3 *1ft/12in = 2360 ft*kip
V_u = (2360ft*kip/25in+2360ft*kip/25in)*12in/ft = 2266 kips
$t_{p\ req'd}$ = (2266kip-231kip)/(0.9*0.6*50ksi*14.5in) = 5.20 in for one plate

Req'd doubler plate thickness based on seismic force w/ R=1.0 (see C11.2A Pg. 171):
M_u = 3.5*94.2 ft*kip = 330 ft*kip
V_u = (330ft*kip/25in+330ft*kip/25in)*12in/ft = 317 kips
$t_{p\ req'd}$ = (317kip-231kip)/(0.9*0.6*50ksi*14.5in) = 0.220 in for one plate

FIGURE 1 Web Doubler Plate Analysis

Example No. 3

The building is a two-story retail structure with 28'-0" x 28'-0" bay sizes and 14'-0" floor-to-floor height. The steel frame consists of composite beams and deck on the 2nd floor and steel joists with roof deck for the roof.

Floor beams were generally W16s, girders were W24s, and the columns were all HSS 8x8 sections. The General Notes sheet listed the building as being designed under IBC 2006 and the AISC 2005 seismic provisions. The seismic lateral load-resisting system was listed as ordinary concentric brace frames with R = 3.25.

There were several braced frames in each direction of the building along the exterior walls. These were single diagonals across the 28' bay width approximately 31 feet in length. The brace sizes were listed as HSS 10x10x.625". No brace forces were listed on the drawing.

When the architect saw the drawings, he was concerned since the exterior walls had been detailed using the 8" columns as a basis, and the 14" brace would be sticking out of the wall.

When the steel fabricator started detailing the braces, he was instructed that the braces needed to satisfy Chapter 14 of the AISC 2005 seismic provisions, including Section 14.4.2 “the required strength of the bracing connections is the expected yield strength in tension of the brace, determined as RyFyAg (LRFD) or RyFyAg/1.5 (ASD), as appropriate.” This would result in a connection design force of almost 1000 kips service load at one end to an HSS 8x8 column and W18x35 spandrel beam, and at the other end to an HSS 8x8 column with a 10"x14" baseplate.

When queried about the member, the engineer was confident in his design, as he had used a “state-of-the-art” computer program, and the analysis and design were “exact”, in his words. When pressed by the peer reviewer to show the computer model, the error soon became apparent.

The computer program had been set to limit width to thickness ratios (b/t) and slenderness ratios for all brace members within the seismic load resisting system based on the AISC 1997 seismic provisions. The program had not been updated to reflect the AISC 2002 or 2005 provisions, nor had the engineer recognized that the slenderness requirements do not apply to single diagonal braces. Refer to Figure 2 for a history of changes in AISC slenderness and b/t ratios for OCBFs since 1992.

Once this error was pointed out, the braces were re-designed to HSS 8x8 members with several hundred kips connection capacity, and the Architect and steel fabricator were both happy.

OCBF BRACING REQUIREMENTS			
AISC SEISMIC PROVISIONS			
DATE	REFERENCE	SLENDERNESS	LOCAL BUCKLING
1992	Section 9.2	$\frac{L}{r} \leq \frac{720}{\sqrt{Fy}}$	$\frac{b}{t} \leq \frac{110}{\sqrt{Fy}}$
1997	Section 14.2	$\frac{KL}{r} \leq \frac{720}{\sqrt{Fy}}$	
	Table I-9-1		$\frac{b}{t} \leq \frac{110}{\sqrt{Fy}}$
2002	Section 14.2	$\frac{KL}{r} \leq 4.23\sqrt{\frac{E}{Fy}}$ (For V or inverted V frames)	None?
2005	Section 14.2	$\frac{KL}{r} \leq 4.23\sqrt{\frac{E}{Fy}}$ (For K, V or inverted V configurations)	
	Table I-8-1		$\frac{b}{t} \leq .64\sqrt{\frac{E}{Fy}}$

FIGURE 2

Lessons Learned

These are only a few examples of mistakes and errors which are taking place almost everyday in design offices. Even though these examples are taken from a relatively short timeframe of 2008 to 2009, the applicable Codes and referenced seismic provisions run from IBC 2000 to IBC 2006 and correspondingly AISC provisions of 1997 to 2009. Thus, clearly one of the contributing factors to the errors being made is the number of changes in the Codes and provisions, and for many small practices which may design only one or two steel buildings over several years, it is difficult to stay up with the changes.

It is likely the same occurs with fabricators, as the larger firms with detailing and engineering firepower stay abreast of the Code changes and provisions and understand the cost impact to their connection design and fabrication. Smaller fabricators, with modest detailing capability and no engineering support, are more vulnerable to cost impacts, especially if being exercised by an engineer who is insisting on the prescriptive provisions.

Code writers and Reference Standard writers could do a better job in this regard as well, since there has been sufficient documentation and countless nightmare stories versed mostly by fabricators of the connection design and cost problems. If, for example, the prescriptive requirements were reversed to call for supplying the connection design force and then default to full capacity, perhaps many more engineers would implement their usage, fabricators would be

able to provide for reasonable bids and connection designs, and Owners would ultimately benefit with a more economical building.

The authors recognize that oftentimes Code writers take a position of making provisions "dummy-proof", and thus the prescriptive provisions err on the conservative side for this reason. However, Code writers should also consider that Owners want economical and efficient structures, and they have long memories about complaints, change orders and delays which occur where the connection design of a steel building bogs down. For this reason, it is important for design engineers to view the use of prescriptive provisions as a last choice instead of a convenient starting point.

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