Building Design for Moderate Seismic Regions

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INTRODUCTION

The authors’ firm practices in the northeast, where steel frames are routinely more cost-effective than cast-in-place concrete frames, and ordinary concentric braced frames are the preferred lateral force resisting system where architectural constraints allow. This paper discusses how Building Code and Reference Standard revisions have affected the design, detailing, fabrication and erection of these preferred structural solutions, and how research into the influencing factors behind Code revisions enhances cost-effective solutions.

The design case studies are all taken from the Commonwealth of Massachusetts which, for the time period of 1997 to 2008, permitted construction projects under the 6th Edition of the Massachusetts State Building Code, which is based on BOCA 1997 (ASCE 7-95) and certain prescriptive exceptions which result in most buildings being designed for Seismic Design Category C, including special seismic detailing consistent with the 1992 AISC Seismic Provisions. Spectral design acceleration values for Massachusetts under the 6th Edition are based on: \( Sa = 0.12 \times \frac{T}{H}^{0.75/50}; \) Site Factors = 1, 1.2, 1.5, and 2. Wind forces in the Mass Code 6th Edition are based on fastest-mile wind speeds of 90 mph with standard exposure coefficients and no breakdown of leeward and windward forces.

During the 1997 – 2008 time-frame, National Code Standardization and updates were becoming almost uniform with two influences:

1. The amount of damage from the Northridge Earthquake, both to buildings and infrastructure, and the development of the new USGS Seismic Hazard Maps through NEHRP, led to national adoption of seismic design in low and moderate hazard regions.
2. Market forces pushing for standardization allowed for generally uniform adoption of the International Building Code with planned updates every 3 years. IBC 2006 included new USGS seismic maps with significant changes in ground accelerations, and also new R Factor tables with significant changes, including the substantial change of R from 5 to 3.25 for Ordinary Concentric Braced Frames.

Thus, while surrounding New England States adopted wholesale more current Codes and Reference Standards, the Massachusetts Code, with its extensive amendments to former Reference Standards, fell several versions behind. This disparity between states required careful consideration of design and reference standard usage, depending on the jurisdiction where the design was being constructed.

Also, during this time period, the Authors’ firm and Tufts University Graduate Students were collaborating on research studies related to seismic performance of building structures in low-to-moderate seismic regions [Sorabella 2006, Nelson 2007, Appel, 2008, Gryniuk 2008]. Keeping research current with changing Building Code and AISC Reference Standard provisions was challenging, but also beneficial, as the research spurred activities, correspondence and dialogue,
with insight into new and planned revisions [AISC 2008]. During the preparation of this paper, Massachusetts adopted a 7th Edition Code based in part on IBC 2003, but including ASCE 7-05 Seismic Hazard Maps and R Factors. What follows is a description of building frame designs during this period, and how they were influenced by these conditions. Figure 1 shows frame elevations for the 6 buildings discussed.

FIGURE 1 – BUILDING ELEVATIONS
CASE STUDIES

Wentworth Institute of Technology – Dormitory

The Wentworth Institute of Technology Dormitory project is an eight-story residential building with a gabled mechanical penthouse, pictured in Figure 2. The building is clad with brick veneer and a signature precast concrete eyebrow. The site in Boston’s Back Bay is a filled site classified by the Geotechnical Engineer as $S = 1.5$. The building has a partial basement for mechanical equipment, and the foundation and super-structure are supported on the stiff clay crust beneath the Back Bay fill.

![FIGURE 2 – WENTWORTH INSTITUTE OF TECHNOLOGY, STUDENT RESIDENCES](image)

The steel-framed structure is a traditional solution to the double-loaded corridor dormitory with interior columns on each side of the corridor aligned with perimeter columns and bay spacings of approximately 25'-4" x 23'-0", with an 8'-0" spacing at the corridor. Floor construction consists of 3¼" LWC slabs on 3" metal deck supported by composite steel beams at 11'-6" on center. Perimeter beams supporting brick and/or precast veneer are designed for stringent L/600 deflection criteria.

Lateral loads are resisted by OCBF in inverted V patterns located in demising walls between the rooms in the short direction and along the corridor wall in the long direction. The gable roof penthouse afforded the opportunity to use a hat-truss in the long direction, thereby reducing the total number of braces.

Using $S_a = .12g; S = 1.5; R = 5$, and the Code-prescribed period $T = 1 \text{ sec } +/-$ in each direction, the $C_s = 0.045$, resulting in base shears of 400K in each direction. Wind load forces
were somewhat less for a 90 mph (fastest mile) and Exposure C; V short = 370K; V long = 110K. This building and framing system is representative of designs prepared under the 6th Edition of the Massachusetts State Building Code and likely those designed under IBC 2000 in Seismic Design Category C.

**Wentworth Institute of Technology Dorm R = 3 Study**

An early study was conducted to assess framing using an R=3 braced frame alternative with no seismic detailing [Gryniuk and Hines 2004]. It should be noted that an R=3 design was not technically legal under the 6th Edition of the Massachusetts State Building Code, but it offered a good comparison to the traditional R=5 solution. Two full sets of drawings were prepared and priced by two fabricators active in the northeast.

The findings of this study were somewhat surprising to the industry, but not necessarily to the authors, in that they showed that the R=3 design was not only less expensive, but also lighter. The surprise came from conventional thinking at the time which assumed that using the lower R factor resulted in higher (conservative) forces which offset the requirement for seismic detailing. In fact, since an R=5 design with seismic detailing calls for a $\Omega=2.0$ design and with prescriptive KL/r and b/t requirements for bracing members, it became obvious that the R=5 design would be heavier and have more costly connection designs as well.

**Northeastern University Building A**

The first of eight buildings in Northeastern University’s new West Campus was a serpentine-shaped dormitory with a 6-story wing and a 13-story wing, pictured in Figure 3. Structural steel offered the ideal solution for the varying plan geometry and a variation of the Wentworth solution was applied as this building also had a double loaded corridor.

![FIGURE 3 – NORTHEASTERN UNIVERSITY WEST VILLAGE RESIDENCES, BUILDNG A](image-url)
While the Northeastern site is only several thousand yards from the Wentworth site, the foundation conditions vary. The clay layer is closer to the surface, such that spread footings can be used, but the clay beneath the stiff crust is thicker and softer, resulting in the Geotechnical Engineer recommending a site factor of 2.0. Doubling the seismic forces for site conditions is challenging when trying to design cost-effective solutions, especially with a 13-story wing and moderately heavy (brick façade) building.

Thus the proposed lateral load system of OCBFs using $R=5$ included arranging the braces to orthogonally align with a shared column to mobilize as much two-way action and gravity load as possible to resist the seismic overturning forces. The 13-story wing required a basement for mechanical equipment and to ‘unload’ the clay for replacement by building weight. The perimeter columns forming parts of the short direction braced frames could mobilize the full depth foundation wall, strip footing, and soil thereon for overturning resistance. By pairing short direction braces along the same column line on either side of the corridor, the interior columns with opposite direction vertical reactions from overturning forces were supported on the same foundation element, thus eliminating the need for substantial net gravity load resistance. Similar to the Wentworth structure, this building was considered very light when measured against market standards, albeit the piece count (i.e., pieces/ton) was considered somewhat high.

FIGURE 4 – NORTHEASTERN UNIVERSITY WEST VILLAGE RESIDENCES
Northeastern University Building G

This was the sixth building added to Northeastern’s West Campus, and again the shape was somewhat non-traditional. Part of the plan characteristics and position of the buildings in the West Campus had to do with phasing plans -- older buildings were being worked around while occupied and then demolished once the new structures were complete, while also conforming to the overall master plan and the Architect’s flourish.

Building G is a 6-story dormitory with a double-loaded corridor, but a new twist on this building was the client’s desire to add several seminar rooms at the lowest levels. This building can be seen near the top center portion of the aerial view shown in Figure 4. located Of course classroom bay sizes and ceiling heights are not consistent with typical bay sizes reserved for residences, and thus the traditional solution used at Wentworth and Northeastern’s Building A was modified.

Since market pressures at the time suggested that erection costs were increasing faster than material costs, and erections costs were tied directly to piece-count, consideration was given to eliminating one of the paired columns along the corridor. The notion to eliminate one column was tested for feasibility of typical floor beam over the longer space from spandrel to the far side of the corridor, approximately 29 feet, and to rely on a single line of bracing to resist overturning. Both conditions proved feasible within an economic range, and since certain corridor columns would need to be transferred for the larger bay-sized classrooms at Level 1, eliminating one row of corridor columns reduced the number of column transfers.

The building solution is an important reminder that solutions are rarely “cookie cutter” as changing Code provisions, changing market conditions, and changing Owner requests bring continuing revisions to what at first may appear to be standardization.

Boston W Hotel

The W Hotel in downtown Boston is a 26-story residential tower with low-rise hotel rooms and upper-story condominium units, shown under construction in Figure 5. Originally conceived using a staggered truss approach, market conditions presented by the construction manager at the time suggested that a conventional steel-braced frame would be more cost effective.

While the plan geometry with bay sizes of 28'-0" x 27'-0" and a double-loaded corridor easily adopted a conventional composite floor framing layout, the building height at 310 feet could not be economically resisted with single bays or even opposing bays of braces in the short direction without some mechanism which would mobilize the full width of the building.

This mechanism appeared in the form of a rigid frame formed by the columns bordering the corridor. By moment connecting the floor beams crossing the corridor, and studying the I/L of girders and I/H of columns, proper ratios of stiffness could be provided which would allow the two adjacent braces to act as a single line.
Further advantages to the system were discovered during detailing discussions with the fabricator based on the idea of a ladder column, where the two columns and corridor floor beams were fabricated in the shop. The advantages were less expensive, and better quality-controlled moment connections were made and tested in the shop, resulting in a simplified moment connection for the bottom flanges of the beams, since the backing bar would not need to be removed and back-gauged, and a single piece could be shipped and erected.

Northeastern University – Parcel 18

The Northeastern Parcel 18 project is a 1200-bed dormitory where three residential towers, varying from 22, 19 and 9 stories, surround a two-level dining hall with a landscaped roof. The fourth edge is closed-in by a six-story office building. Figure 6 shows the building under construction.

As with the W Hotel, the initial concept for the residential towers was a staggered truss. This project had extremely aggressive goals for floor plate efficiency using a 9'-4" floor-to-floor height and 18'-0" x 19'-0" bay sizes. Initial studies of the staggered truss showed the steel frame with precast plank floors to be very efficient; however, with the staggered truss clear-spanning, the full 47'-0" width of the building, the high column loads would require drilled caisson shaft foundations. The shafts would need to be over 150' in depth, and thus a traditional braced frame solution, which could rest on a mat foundation, was recommended.
While this recommendation was well-founded with respect to foundation, schedule and cost considerations, the aspect ratio for the towers (such as the 22-story building rising 256' with a 47' width) at greater than 5 to 1 would soundly test an economical steel frame solution, and again the site factor was $S = 2.0$.

Drawing on the advantages discovered with the W Hotel from the braced frames, we again looked at a rigid ladder column to mobilize the full 47'-0" for overturning. In addition, alternate bracing members and connection types were introduced based on ongoing graduate student research [MSC 2008]. These details can be seen in Figure 7. In lieu of using slotted HSS brace members, back-to-back HSS members were used. While HSS braces are cost effective and have certainly been the most common brace type since the advent of seismic detailing provisions, the actual slotted brace-to-gusset connection is prone to vulnerability when scrutinized. First the slot is cut and the heat from the cutting wants to close the gap. Oftentimes the slot is enlarged in the field because of fit-up problems. Thus the area at the root of the slot may have multiple series of induced residual stresses, making it more vulnerable to fracture, especially if the connection is not reinforced. Using back-to-back HSS sections with stitch plates is simpler to fabricate, simpler to erect, less likely to need connection reinforcement, and only slightly heavier than a single HSS.

Also considered for connection design was the use of an end plate beam with brace and gusset connections to the columns in lieu of the traditional double-angle connected gusset and
beam. The goal was to be able to design for a specific amount of floor beam axial load transfer force and supply some post brace buckling moment frame capacity with the connection.

While advancing the studies used in the W Hotel project for Parcel 18, and attempting to reduce cost, it was discovered that every other opposing brace could be eliminated while still preserving the same overturning resistance. Since shear force is modest compared to overturning demands, and since brace sizes are often controlled by KL/r and b/t ratio requirements, every other brace could be eliminated without almost any impact to other members. While this creates a transfer force requirement at every floor across the corridor frame, this is accomplished fairly simply with the end-plate-connected beam and the moment-connected ladder column.

The resulting structure was extremely cost effective as was the fabrication schedule. Erection was completed almost one month prior to the original schedule, proving that very light buildings, even with a high piece-count, will erect quickly. The project successes include the fact that the steel fabricator, Novel Iron of Greenland, New Hampshire acknowledged the importance of some of the innovative factors used in the design, and agreed to supply and fabricate several representative beam-to-column units for testing at the University of Illinois as part of ongoing research for effective steel frame designs.

**Northeastern University Building K**

Building K is a 300-bed Dormitory on the east side of the campus and features multi-curved faces and a central core. This would be the first building on the Northeastern Campus to be designed under the 7th Edition of the Mass Code with the changes as described above. With a planned building height of 18 stories totaling 225' the quarter-moon shaped core would not be sufficiently rigid to carry all of the lateral loads economically. Furthermore, there were

**FIGURE 7 – NORTHEASTERN UNIVERSITY PARCEL 18, PARTIAL BRACING ELEVATION**
numerous architectural and mechanical access paths preventing the perimeter walls of the core to be effectively braced.

Since this dormitory was a “suite” design, limited door openings were required off the corridor to the rooms. Thus the corridor wall proved a far better location for the bracing system, and it also gave a larger footprint for overturning. To avoid the use of eccentric-braced frames at the doors, ladder columns were used at each door opening.

To reduce overturning forces, several outrigger trusses were added in the demising walls between suites. The resulting frame is extremely efficient despite the complex plan geometry. Impacts from the new Code provisions were extensive from a design point of view, but minimal from a building cost point of view, since the lower R factor (3.25 vs 5.0) was offset by the lower ground acceleration.

CONCLUSIONS

Six buildings of similar occupancies were designed and constructed during a ten-year period during which Code revisions and market conditions were both volatile. The Case Studies demonstrate the importance of researching and understanding where Code changes are coming from and where they are headed. It is important to embrace the concepts behind these changes in order to provide cost-effective and sound designs. The experience also suggests that fairly large changes have occurred to the R Factors, seismic detailing requirements, site factor contributions and even spectral accelerations. Thus, when designing in low and moderate seismic regions, system performance should be measured as much by fundamentally sound principles as by “so-called” exact analysis.

REFERENCES