

Simulated Performance of Steel Moment-Resisting Frame Buildings in the 2003 Tokachi-oki Earthquake

Thomas Heaton, Jing Yang and John Hall

Earthquake Engineering Research Laboratory, California Institute of Technology

Abstract

We simulate the response of 6- and 20-story steel moment-resisting frame buildings (US 1994 UBC) for ground motions recorded in the 2003 Tokachi-oki earthquake. We consider buildings with both perfect welds and also with brittle welds similar to those observed in the 1994 Northridge earthquake. Although existing short, strong buildings in Japanese towns performed well in this earthquake, our simulations indicate that flexible buildings would have been strongly excited by this earthquake. Simulated deformations are large enough in some basin regions that one could expect irreparable damage at many locations for both the 6- and 20-story buildings. In a few instances, the 20-story building with brittle welds experienced dangerously large deformations.

Key words: Tokachi-Oki Earthquake, Strong ground motions, High-rise building, Inter-story drift, Nonlinear, Simulation

1. Introduction

Despite the proximity of the 26 Sept. 2003 Tokachi-oki earthquake (M_w 8.3) to the island of Hokkaido, damage was relatively mild. Although there are numerous coastal towns in southeastern Hokkaido, there are no large cities within the region of heaviest shaking. The Tokachi-oki ground motions are especially notable because of their large long-period components (Koketsu *et al.*, 2005). The short, stout buildings of Japanese coastal communities performed well in this long-period shaking. The most serious damage was due to sloshing in petroleum storage tanks located about 150 km west of Cape Erimo, which is the peninsula that juts out over the rupture surface (Hatayama *et al.*, 2004). How would tall, flexible buildings have performed in this great earthquake?

2. Ground Motions

The 2003 Tokachi-oki earthquake is far and away the best recorded great subduction earthquake. We have been analyzing ground motions records from this earthquake for the past several years and a comprehensive description of the data and its proc-

essing to ground displacement can be found in Clinton (2004).

Figure 1 shows the location of K-NET and KiK-net stations that recorded the 2003 event. We carefully analyzed each record in an attempt to recover the static ground displacement. We used the methodology of Boore (2001), combined with GPS data, to best determine the static displacements. Figure 2 shows horizontal ground displacements at selected stations.

3. Building Simulations

Although short, stiff buildings performed well, an obvious question is raised by this event. How well would tall, flexible constructions perform in these long-period ground motions? In order to answer this question, we simulated the response of 20-story and 6-story steel frame buildings using each of the 276 ground motions recorded on Hokkaido Island. These buildings were designed by Caltech Prof. John Hall according to the 1994 Unified Building Code (UBC94) assuming seismic zone 4 and soil site S2 (corresponding to soil type C in UBC97).

The buildings are symmetric, which allows us to

* e-mail : heaton@caltech.edu

model their response using 2-D models. The seismic simulation finite element program was developed by John Hall (1997) and it is based on a fiber-element model that includes material nonlinearity as well as geometric nonlinearity (P-delta and buckling). In modeling the beam-to-column connections and col-

umn splices, we can simulate the fracture of welds. The natural periods of the 20- and 6-story buildings are 3.5 sec. and 1.5 sec., respectively. Figures 3 and 4 show pushover curves for the 20-story and 6-story buildings, respectively.

Horizontal loads (given as a percentage of the building weight) are applied statically and the roof displacement is calculated as a function of the load. The blue lines correspond to buildings with perfect

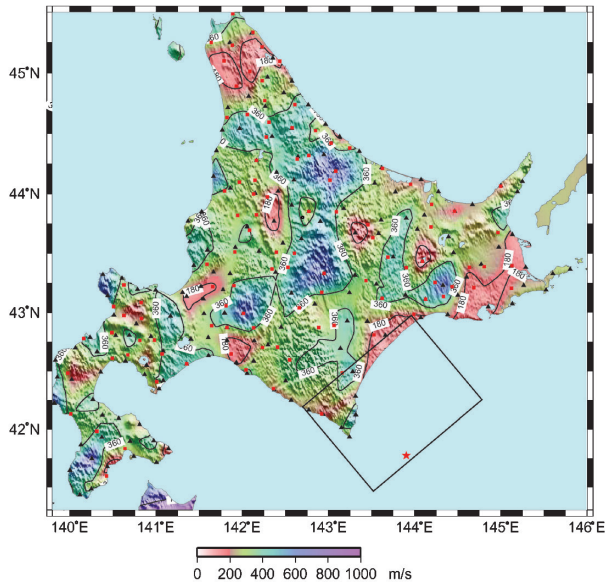


Fig. 1. The locations of K-NET (black) and KiK-net (red) stations that recorded the 2003 Tokachi-oki earthquake (M_w 8.3). The approximate surface projection of the rupture surface is given by the black rectangular box (Koketsu, 2005). Approximate shear-wave velocities in the upper 30 meters are contoured.

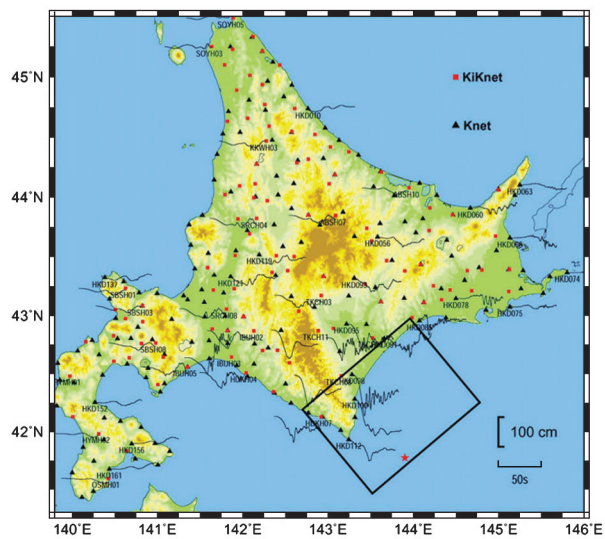


Fig. 2. The radial components of ground displacement for selected stations; the largest recorded displacement was 1.5 meters.

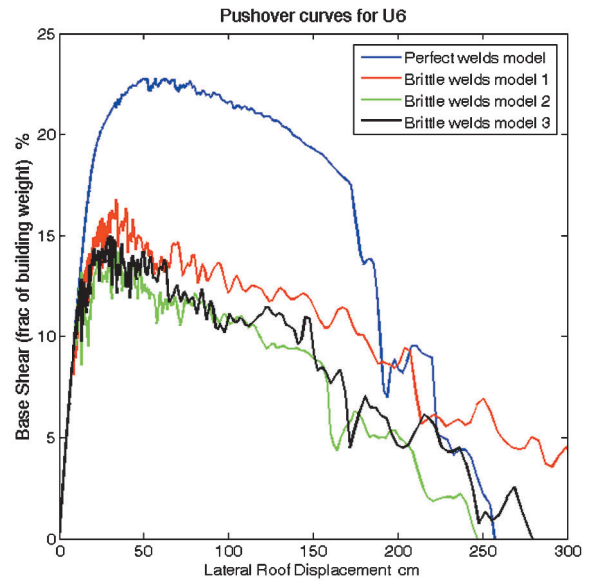


Fig. 3. Pushover analysis of the 20-story steel frame building designed to the 1994 US Unified Building Code.

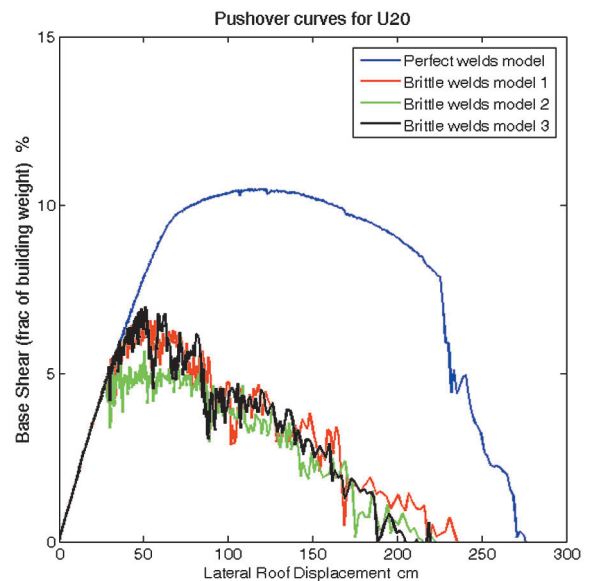


Fig. 4. Pushover curves for the 6-story steel frame building (US UBC).

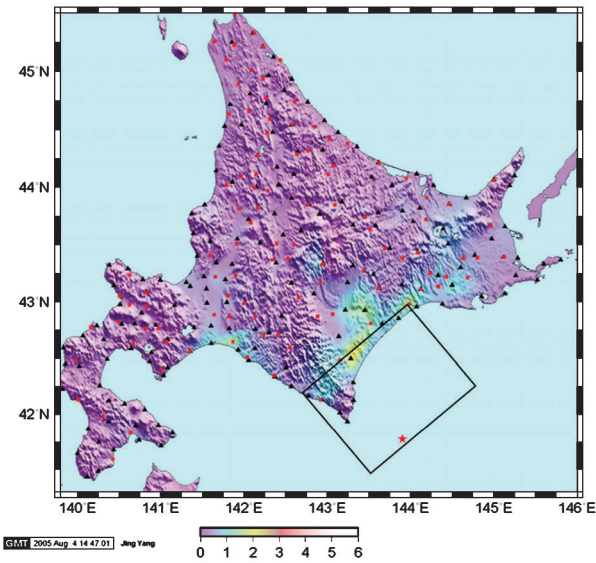


Fig. 5. Maximum inter-story drift (in percent) for the 20-story steel frame designed to the US code and assuming perfect welds. The building response is approximately elastic when the drift is less than 0.5%. Drifts in excess of 2% are considered to be quite serious and buildings may be damaged beyond repair at these locations.

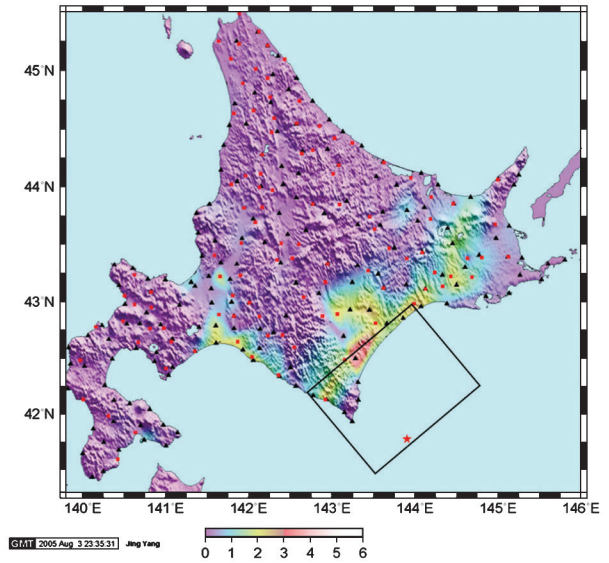


Fig. 6. Maximum inter-story drift (in percent) for the 20-story steel frame designed to the US code and assuming brittle welds.

welds, whereas the other lines correspond to buildings with welds that fracture randomly when local strains exceed a fracture criterion that is compatible with the observations of welds in the 1994 Northridge earthquake. The presence of brittle welds decreases the global yield strength of the buildings. In the case of the 20-story building, brittle welds significantly reduce the ductility of the building (*i.e.*, the ultimate deformation that it is capable of achieving) because of the importance of P-delta effects for tall buildings.

Figures 5 and 6 show contours of the maximum inter-story drift (relative horizontal displacement of adjacent floors divided by the story height, *i.e.*, basically the building shear strain) for the US-code 20-story buildings assuming perfect welds and brittle welds, respectively. Drifts of less than 0.5% (the purple colors) are considered to be in the elastic range of the buildings. Notice that ground motions directly above the rupture surface at Cape Erimo cause less simulated damage than stations located in a band just to the northwest. This seems to be an effect of the local site geology. That is, there is a correlation between simulated building damage and local shear-wave velocity as indicated in Fig. 1. Fig-

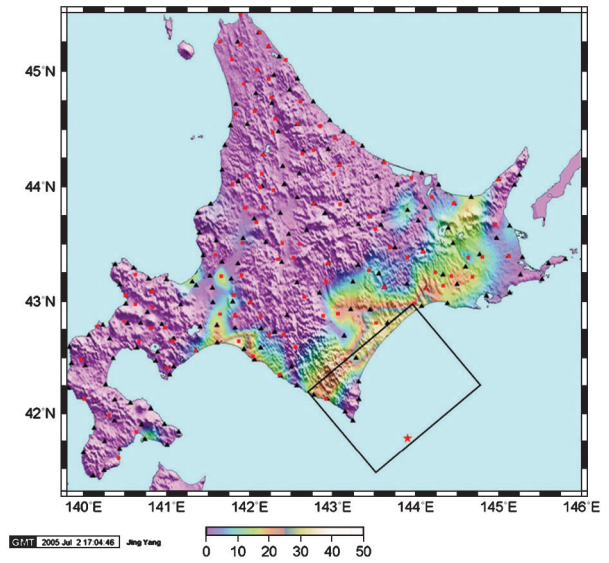


Fig. 7. Percentage of the total welded moment-resisting connections that fractured for the 20-story US building.

ure 7 shows the percentage of welded moment resisting connections that fractured in each of the simulations for the 20-story building.

Figures 8 through 10 show the drifts and weld fractures for the US-code 6-story building.

In order to better clarify the collapse question associated with these simulations, we introduced a new parameter that we call the “safety factor.” This

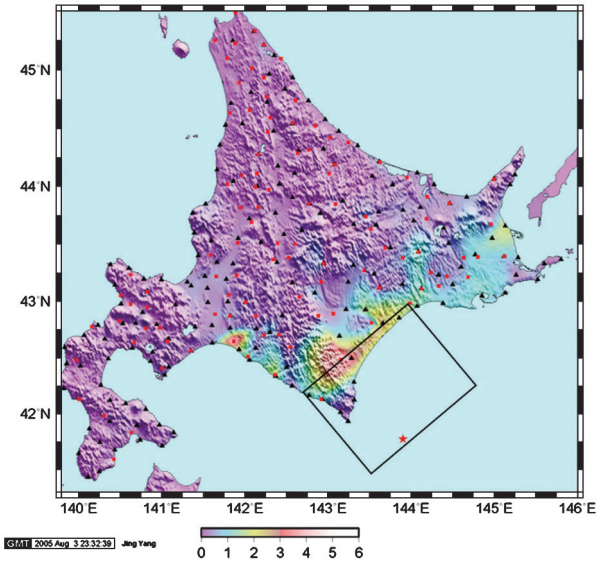


Fig. 8. Maximum inter-story drift (in percent) for the 6-story steel frame designed to the US code and assuming perfect welds. The building response is approximately elastic when the drift is less than 0.5%.

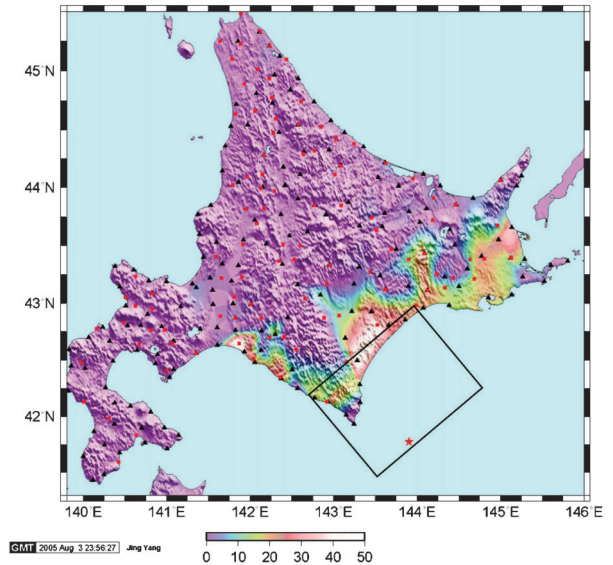


Fig. 10. Percentage of the total welded moment-resisting connections that fractured for the 6-story US building.

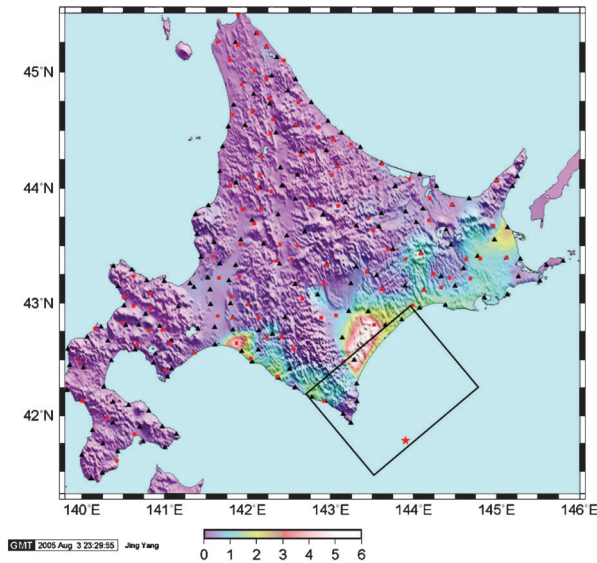


Fig. 9. Maximum inter-story drift (in percent) for the 6-story steel frame designed to the US code and assuming brittle welds. These drifts are larger for the 6-story building than for the 20-story building, but the 6-story building is less likely to collapse at these drift levels than the 20-story building which is more sensitive to P-delta effects.

safety factor is defined to be the scalar multiplier of the recorded ground motion that is required to cause collapse of the simulated building. That is, we sim-

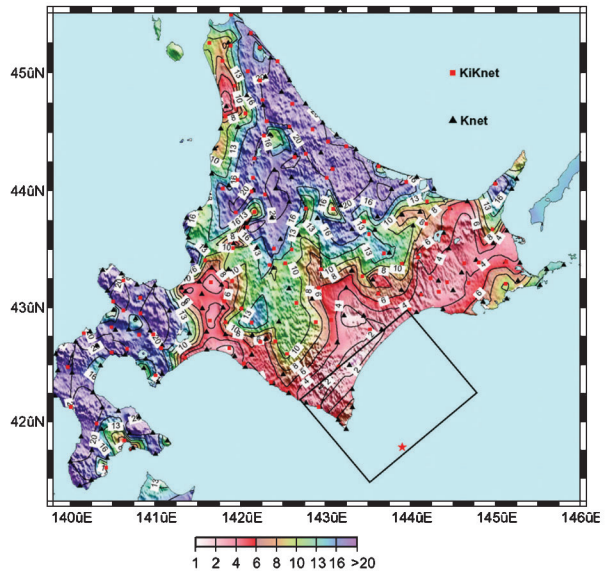


Fig. 11. Safety Factor for the 20-story US code steel building with brittle welds.

ply multiplied the recorded ground motion by a constant and then re-ran the building simulation.

When the multiplying constant was large enough to cause simulated collapse, then we called that multiplier the safety factor; safety factors of larger than 1.0 indicate that the building did not collapse, whereas safety factors less than 1.0 indicate that it did. This safety factor is shown in Figure 7 for the 20-story building. While no buildings showed

collapse in these simulations, increasing the amplitude of station HKD098 by 6% caused simulated collapse. This is well within the uncertainty of this type of calculation.

4. Discussion

Our simulations show that the long-period ground motions recorded in the near-source regions of the 2003 Tokachi-oki earthquake would have caused large inter-story drifts in flexible steel moment-resisting frame buildings designed according to the US 1994 UBC.

There are significant regions in which both 6- and 20-story buildings would have been seriously damaged with inter-story drifts exceeding 1%. Although none of the buildings collapsed in these simulations, some of the 20-story buildings were close to collapsing. Although inter-story drifts were actually larger in the 6-story building than they were in the 20-story building, the 20-story buildings were actually closer to collapse as can be seen by studying the pushover analysis shown in Fig. 3.

The pernicious problem of brittle welds was discovered in the aftermath of the 1994 Northridge earthquake (FEMA, 2000). Prior to this time, it was assumed that mild steel always deforms in ductile manner. However, inspection of many steel buildings indicated the brittle failure of many welds before large plastic strains developed. Our simulations indicate that buildings with strong welds performed significantly better than those that were assumed to have brittle welds.

While none of our simulations indicated the collapse of our structures, some simulations were close to collapse. It seems clear that the low-rise, strong buildings that actually experienced this earthquake were better suited for this type of long-period ground motion.

We are currently performing similar simulations on 6- and 20-story buildings that are designed according to the Japanese building code. These buildings are typically about 30% stronger (base-shear yield strength) than comparable US buildings, but they

come with the penalty of greater stiffness, which tends to increase the forces in a building for a given ground motion.

While the 2003 Tokachi-oki ground motion data set is far and away the best data set available for a great subduction earthquake, it is important to recognize that long-period ground motions may be different in other great earthquakes. That is, many factors control these long-period ground motions (especially the slip history). Therefore, there is much that we do not know about how high-rise buildings will perform in future great subduction earthquakes.

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