Understanding the basics of earthquake design

Knowledge of the *Ontario Building Code* formulas is only part of the story when designing earthquake-resistant buildings. A thorough understanding of current theories of earthquake-resistant design is equally important.

BY DAVE TIPLER, P.ENG.

The 1997 *Ontario Building Code* provides relatively simple formulas for calculating earthquake loads. Using these formulas blindly without understanding current theories of earthquake-resistant design, however, will, more than likely, result in buildings that are inadequate to resist code-specified earthquake forces. Without a grasp of the basics, designers may become hopelessly lost in the myriad numbers and formulas presented in the latest building codes. Over the past few years, I have gathered information on earthquake design, and want to share a simplified interpretation of it so that a better understanding of earthquake design will become more universal. The discussion pertains to single-storey steel buildings; however, the concepts can be applied to all buildings.

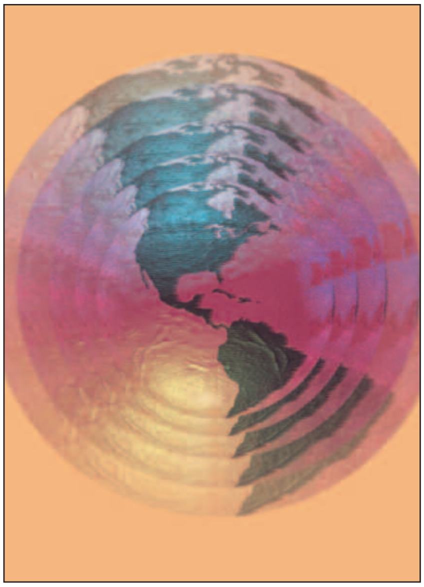
Structural consultants are familiar with the formula Ve=vISFW, where Ve is the equivalent static seismic force representing elastic response. In other words, in the event of a worst-case, code-specified earthquake, a building designed with a lateral load-resisting system that does not yield will experience an equivalent static horizontal force of Ve. All of the building components will experience forces corresponding to this horizontal force. Actually, the authors of the code threw in a calibration factor of 0.6 to bring theory more into line with real-life observations. If a building responds in an elastic manner under full earthquake force, as defined in the code, it will, for design purposes, experience an equivalent static horizontal force of 0.6*Ve.

The above discussion should be clear enough, but here is where things get a little tricky. The *Ontario Building Code* appears to allow designers to reduce the horizontal earthquake force by a factor of R, depending on the type of lateral load-resisting system chosen for the design. This is implied by the formula: V=0.6*Ve/R. In the 1997 *Ontario Building Code*, the value of R can vary from 1 to 4, depending on the chosen lateral load-resisting system. The code defines 0.6*Ve as the force associated with elastic response, and it then allows design for lateral load-resisting systems for forces that can be as little as 25 per cent of 0.6*Ve, if we choose, for example, to use a ductile moment resisting frame. If we design a building using a concentric-braced frame with nominal ductility, we can use R=2, i.e. the building will resist half of the potential earthquake force. Is this some sort of voodoo magic? It doesn't appear logical that we can alter the forces of nature by simply choosing a specific type of lateral load-resisting system.

In fact, there is no voodoo at work here, and we are not changing the forces of nature. What we are doing is designing a lateral load-resisting system that yields at a predetermined horizontal force, which can be as little as 0.6*Ve/R. You may now be thinking: "Did he just say the structure is going to yield? That doesn't sound like a good thing!"

And, you're right, structural engineers typically don't want things to yield. Under true static loads, such as gravity loads, designing structures to yield at loads less than the applied loads would result in disaster. But we must remember that earthquake loads are not static, reversing within a very short time. Therefore, yielding of lateral load-resisting systems can occur without resulting in the collapse of a structure. Whether you realize it or not, that is exactly what you are doing when you use an R factor greater than 1 for earthquake design.

Once you have come to grips with the above concept, you have to decide which part of the structure you want to design to yield, so that yielding can occur in a controlled manner during an earthquake or other violent event. Elements such as roof diaphragms, columns, footings, vertical braces, and vertical brace connections are the basic structural components that make up the lateral load-resisting system in a single-storey steel building. For various reasons, yielding in the vertical braces is generally preferable. From here on, I will refer to all of the other components in the lateral load-resisting system as brittle elements.



Much like a fuse in an electrical circuit that protects the wire and the load from accidental overload, properly designed vertical braces yield before damage is done to the brittle elements of the structure. This is where fatal mistakes can be made if you do not fully understand the current theories of earthquake-resistant structures. If we choose, for example, to use a lateral load-resisting system that has a code-specified R factor equal to 2, and we proceed to design the brittle elements for forces resulting from V=0.6*Ve/2, we must design the vertical braces to yield precisely when a force of V is applied to the structure, not more. Just as we would not want to install a 20-amp fuse in an electrical circuit that could be damaged with 15 amps of current, we cannot have vertical braces that can transmit overload to the brittle elements in our structural circuit. If the vertical braces yield at loads greater than those generated by V and the brittle elements have been designed for the loads generated by V, uncontrolled failure in at least one of the brittle elements will result. In fact, we should always design the brittle elements for a horizontal force level that is somewhat higher than that required to yield the braces, to ensure the vertical braces yield before the brittle elements fail. As such, the R factor should not be considered as an absolute value, but rather a maximum value that can be used to reduce earthquake forces for the purposes of designing our structural fuse (the vertical braces).

This idea can be found in the CSA code requirements. For example, the CSA S16.1-2000 edition states in clause 27.4.4.2 that vertical brace connections must be designed for AgFy of the vertical brace. This provision is in the code to ensure the brace yields before the connection fails, thus eliminating the possibility of uncontrolled failure of the connection. The latest edition, CSA S16.1-2005, has an even better description of earthquake design theory in Section 27. Do yourself a favour and read it.

If you have understood the above discussion, you will start to see all sorts of real-life design problems. Sometimes, the vertical braces have to be over-strengthened for other reasons. Minimum slenderness ratio requirements for diagonal braces may require an increase in the cross-sectional area of the braces, which will increase the yield force in the brace. In this case, brittle elements cannot be designed using R=2.

It is easy to make the mistake of believing that wind governs when the wind load is greater than V=0.6*Ve/R. However, say we have a factored horizontal wind load of 100 kN, and have calculated 0.6*Ve to be 150 kN. We choose to use a concentrically braced frame with nominal ductility, hence R would be a maximum of 2. V would then become 150/2=75 kN. Some designers might be tempted to say that wind governs, and ignore earthquake requirements. In fact, if the braces were designed to yield precisely when the structure experienced a load of 100 kN, wind loads and earthquake loads would be equal. If a designer provided braces that resist a horizontal load greater than 100 kN (and most designers would when designing for wind), earthquake loads would govern, because the vertical braces act as the structural fuse in earthquake design. If you over-strengthen the fuse, you must increase the strength of the brittle elements accordingly. So the only time wind governs over earthquake is when the wind load is greater than 0.6*Ve.

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Another real-life problem with which structural engineers have to deal when designing buildings for earthquakes is the strengthening effects of non-structural building components, such as exterior walls. For example, we go through all the painstaking calculations to make our vertical braces yield at specific load levels to protect the roof diaphragm we have designed for reduced earthquake loads. The precast concrete wall supplier then rigidly connects a multitude of precast concrete panels to the perimeter of the structure, thus ensuring the vertical braces will never be loaded and, hence, never yield. Under full earthquake loading, the diaphragm will probably fail. The 2005 National Building Code of Canada recognized this particular problem (and others), which is why the new code requires precast concrete panels to be connected to main building structures with horizontal sliding connections.

Now, why would we want to go through this process rather than simply design the structure to respond elastically during an earthquake? Simple. If we can design the vertical braces to yield at a lower force than that resulting from 0.6*Ve, we can design less expensive brittle elements. Unfortunately, I have found that for most single-storey steel buildings approximately 6 m high, other code issues typically require vertical braces to be over-strengthened to the point where designing for R>1 is not possible. As such, all components of the lateral load-resisting system have to be designed using R=1, and they can become relatively large. C'est la vie.

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