Impacts of Lateral Code Changes Associated with the 2006 International Building Code and the 2008 California Building Code

By

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ABSTRACT

The 2008 California Building Code (CBC) will adopt the structural section of the 2006 International Building Code (IBC), which includes alterations to the procedure to determine earthquake design loading, and a drastic move to a complicated method to determine design wind pressures. The implementation of the revised 2006 International Building Code, and the subsequent California adoption of the structural section will have significant effects on the design and construction of structures not only in California, but also the rest of the country.

Through a comparison of the design of a steel moment-resisting frame low-rise structure, it was determined that the new code will result in design values that differ from those resulting from the previous codes. In order to compare the relevant codes in different areas of the country, this thesis considers three design scenarios for the low-rise structure: seismic loading in Southern California to compare the 2001 CBC, the 2003 and the 2006 IBC, seismic loading in the Midwest to compare the 2003 IBC and the 2006 IBC, and wind loading in Northern California to compare the 2001 CBC and the 2006 IBC. In the first case, the change from the 2001 CBC to the 2003 IBC was an 8 percent increase in base shear, but a 2 percent decrease from the 2001 CBC to the 2006 IBC. The second case resulted in a 29 percent increase in base shear from the 2003 IBC to the 2006 IBC. The result of the third case was design wind pressures that decreased 20 percent from the 2001 CBC to the 2006 IBC. These design differences will change the design of the lateral force resisting system, especially the later two cases. In addition, the design engineers in California will have to learn a new, greatly more complicated method to design for wind loading. These combined effects of the code changes will impact both engineers and the resulting building designs in all parts of the country.

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1 Introduction

The 2006 International Building Code (IBC) became active in January 2007. As of 2006, 48 of the 50 states used the IBC. This number will increase to 49 by January 2008. California will begin using most sections of the 2006 IBC by July 2007, with full adoption of those sections by January 2008. The structural section is one section of the 2006 IBC that will be adopted by the California Building Code (CBC). The structural section of the 2006 IBC will effectively become that of the 2008 CBC.

The 2006 IBC includes amendments in various areas, but these changes are particularly significant in the seismic and wind sections. The changes in the seismic code that made the 2006 IBC more accurate made it possible for California to safely adopt the IBC, but in doing so the CBC also adopted the IBC's wind code. Effectively, the 2006 IBC is a "meeting in the middle" between the previous editions of the IBC and CBC. Due to this merger, several areas of the country, in addition to California, will undergo both seismic and wind code changes.

Widely known for its earthquakes, California, specifically parts of the Sacramento Valley, will enjoy a seismic code relaxation. The CBC will adopt the procedure detailed in the IBC, and both codes will abandon a seismic zone/category procedure in favor of a more detailed ground acceleration-based approach. The zone/category approach groups together areas that are expected to experience similar ground motions during an earthquake into zones/categories, and then designs structures according to zone/category. The ground motion approach eliminates the grouping of areas and designs structures directly based on expected ground motions. The result of the new method will be a slightly less strict seismic design for most of California, but it will also cause the opposite effect in much of the rest of the country (i.e. more strict designs). The impacts of these changes may turn out to be slight, but they will not be trivial.

The changes in the wind code will affect the country in the opposite way; California will be the area to suffer from a more stringent design procedure. The changes in the wind code between the old and new IBC are few, but California's adoption of the IBC will have significant changes. The wind section of the 2001 California Building Code is based on the 1988 version of the American Society of Civil Engineer's Building Standard, the ASCE 7. This simplified version has been acceptable in California because wind does not tend to control the design of the lateral force resisting system. However, since wind tends to control in other parts of the country, it is important to maintain a more detailed procedure. This drastic change in wind code for California will have profound impacts on not only the structures being designed, but also on the design work of engineers.

Engineering practitioners in California are especially apprehensive regarding the adoption of the wind loading section of the 2006 IBC (which will be the 2008 CBC). Since the design is gaining nearly 20 years of updated code (ASCE 7-88 w/simplifications to ASCE 7-05) at once, the impacts are going to be multi-faceted. It is likely that the more detailed code will be more accurate, but it might also cause some difficulties in design.

Ideally the changes will allow engineers to more accurately and more consistently design structures, however it is counterintuitive that the precautions in parts of the country without a reputation for seismic activity will increase while California's code will become more lenient. In addition, it seems unnecessary to overly complicate the wind code in a state where wind loading does not tend to control lateral designs.

This thesis shall address expected impacts and consequences of the new 2008 California Building Code and 2006 International Building Code from the point of view of a practicing design engineer. Chapter 2 will detail the history and origins of wind and seismic codes, and Chapter 3 will look at case studies that address effects of seismic code changes that have occurred in the past. Since building codes are often difficult to interpret, Chapter 4 presents the codes as they are specified in the 2001 CBC, the 2003 IBC and the 2006 IBC. Chapter 5 includes the three design scenarios and comparisons of the resulting values. Finally, Chapter 6 describes the conclusions and expected impacts of the code changes in both California and the rest of the country.

2 History and Background

As the industrial world advanced, structures became more and more complex and it became clear that designers and contractors would have to improve their traditional methods in order to accommodate these structures. In addition, it became clear that a consistent, standardized set of "special rules and procedures" needed to be compiled and applied to both the design and the construction phases of a project. These special rules and procedures are what we now know as the building code. (Uzumeri 2003)

2.1 Development of Seismic code

Before the 20th century, seismic building codes were based solely on lessons learned from recent earthquakes (Key 1988). Only after a major earthquake occurred was the code updated (Xie 2001). As research in the area advanced, so did the codes, but limited knowledge meant that the building code was little more than an educated guess at what should be designed. Due to this, a building code depended more on designers than on building officials. The effectiveness of a seismic code depends on qualified and accurate knowledge and the subsequent enforcement (Krimgold 1977). Therefore, enforcement of the seismic provisions became a concern over time. Even if perfect, codes cannot provide any protection if not enforced (Uzumeri 2003). Currently, government code-checking agencies review calculations and construction documents, which enforces building codes. These agencies can be at either the state or local lever. For example, the Division of the State Architect in California reviews plans and calculations.

At the beginning of 1990, the Interagency Committee on Seismic Safety (ICSSC), a US federal group, issued an executive order that required federal agencies to follow seismic provisions in federal buildings. This was done in an effort to boost conformity to the seismic building codes. As a result, state and local governments were required to incorporate seismic provisions into their building codes in order to be eligible for federal aid. By 1992 all building codes included the prescribed seismic section. Although a significant step, this order was more concerned with life safety than accuracy, so research and advancements were still required to refine the seismic code.

When designing a structure, an engineer must consider forces that act in all directions. The weight of the building, and other gravity loads (snow, etc) are considered to act in the vertical direction, and although they can act in many directions, other loads such as earthquakes and wind are applied in the horizontal direction. As a result, the vertical loads determine the gravity force resisting system, and the horizontal loads determine the lateral force resisting system. It follows then that seismic building codes aim to predict equivalent lateral loads that a building would experience during a seismic event. The first seismic code was developed in the 1920's and presented the equivalent lateral load, F, using Equation 2-1.

$$F = CW$$

Equation 2-1: Equivalent Lateral Load in the 1920's

where: F is force, in pounds
W is the permanent load of the structure, in pounds
C is the seismic coefficient, usually about 0.10

As research advanced, finding the seismic coefficient became more complex, and it involved other parameters such as soil conditions and the ability of a structure to absorb energy. Despite these advances, some experts believe that the codes do not adequately represent the seismic excitations felt by the structure during an earthquake. (Saunter 1997)

The equation (as of 1997) for equivalent lateral forces during an earthquake is given in the following equation:

$$V = \frac{IZC}{R_W} W$$

Equation 2-2: General Form of Modern Equivalent Lateral Load

where: V is shear, in pounds
I is the importance factor
Z is the seismic Zone factor
R_W is a reduction factor
W is the weight of the structure, in pounds

C is given as a function of the natural period of the structure, T, and a soil amplification factor, S. This relationship can be presented in the same form as Equation 2-1, with the seismic coefficient, C, defined as:

$$C = \frac{1.25 I A_V S}{R_W T^{2/3}}$$

Equation 2-3: Seismic Coefficient

where: A_V is the peak ground acceleration due to the seismic zone, in percent of g, the acceleration due to gravity
I, S, R_W and T are as previously defined

For a 4-story building in seismic Zone 4, the seismic coefficient, C, is 0.092, which is close to the original value set in 1920. (Saunter 1997)

These equations have been further developed and interpreted by building codes. As will be mentioned in the following section on wind codes, contemporary seismic building codes are based on an American Society of Civil Engineer's building standard, the ASCE 7.

2.2 Development of Wind Code

Similarly to earthquake loads, wind loads are considered laterally and contribute to the design of the lateral force resisting system of a structure. Historically, calculating wind loading on a structure combines two branches of science: meteorology and aerodynamics. Predicting wind loads requires both an estimate of maximum possible wind speeds endured by a structure during its lifetime, and an estimate of the resulting pressure exerted on the structure's surfaces. (MacDonald 1975)

Prior to the nineteenth century, structures were designed according to empirical aspect ratios and other proportional rules that had evolved throughout the previous centuries. Wind loading was not a consideration in the design of these structures because it was not necessary to ensure safety. This was true since most major structures were constructed out of stone and other very heavy materials; the weight of a large stone structure alone would mitigate any concerns of overturning due to wind loading. However, as other materials were incorporated into structures, and the overall weights became lighter, structural engineering that considered wind loading became more relevant. (MacDonald 1975)

During the nineteenth century wind speeds, on which wind loadings were based, were predicted using "rules of thumb" based on the effect the wind had on an object that was conveniently on-hand. In 1805, Rear Admiral Sir Francis Beaufort proposed a method to estimate wind forces. This method, shown in Table 2-1, is still used today, although it is no longer based on the performance of sailing a man-of-war, which was a ship used in the early 1800's. (MacDonald 1975)

Table 2-1: Beaufort Scale of Wind Force

<table>
<tr><th>Force</th><th colspan="3">Beaufort Scale</th><th>Velocity* (mph)</th></tr>
<tr><td>0</td><td colspan="3">Calm</td><td>3</td></tr>
<tr><td>1</td><td colspan="3">Light air, or just sufficient to give steerage way</td><td>8</td></tr>
<tr><td>2</td><td>Light breeze</td><td rowspan="3">(or that in which a well-conditioned man-of-war, with all sail set and clean full, would go in smooth water from)</td><td>1 to 2 kn</td><td>13</td></tr>
<tr><td>3</td><td>Gentle breeze</td><td>3 to 4 kn</td><td>18</td></tr>
<tr><td>4</td><td>Moderate breeze</td><td>5 to 6 kn</td><td>23</td></tr>
<tr><td>5</td><td>Fresh breeze</td><td rowspan="5">(or that to which she could carry 'in chase' 'Full and by')</td><td>Royals, etc.</td><td>28</td></tr>
<tr><td>6</td><td>Strong breeze</td><td>Single-reefed topsails and top-gallant sails</td><td>34</td></tr>
<tr><td>7</td><td>Moderate gale</td><td>Double reefed topsails</td><td>40</td></tr>
<tr><td>8</td><td>Fresh gale</td><td>Triple-reefed topsails</td><td>48</td></tr>
<tr><td>9</td><td>Strong gale</td><td>Close-reefed topsails and courses</td><td>56</td></tr>
<tr><td>10</td><td colspan="3">Whole gale, or that with which she could scarcely bear close-reefed main topsail and reefed foresail</td><td>65</td></tr>
<tr><td>11</td><td colspan="3">Storm or that which would reduce her to storm staysails</td><td>75</td></tr>
<tr><td>12</td><td colspan="3">Hurricane, or that which no canvas could withstand</td><td>90</td></tr>
</table>

*Calculated by the meteorological Office and did not appear in Beaufort's original table.

Later in the 1880's the absence of accurate, reliable and safe data prompted the Board of Trade, a US government agency, to issue a ruling that a wind pressure of 56 pounds per square foot be applied to all future structures. This expected loading would be used in calculating the stresses due to wind loading in all structures. At this time, other countries were using comparable values; however, 56 psf is a large amount of pressure, so researchers and engineers began to look for a new method that would lower this value. (MacDonald 1975)

The first large advancement came in 1907, when T. E. Stanton proved that the pressure exerted on a flat plate larger than one square foot was independent of its size. This discovery led to a proposed equation to model wind pressure exerted on the surface of a structure. This equation is shown in Equation 2-4.

$$p = \frac{1}{2} C_d V^2$$

Equation 2-4: T.E. Stanton's Equation

where:
p is pressure due to wind, in pounds per square feet (psf)
C_d is a drag coefficient dependent on site surroundings
V is the max recorded gust speed at the site in miles per hour

The wind code as we currently know it originated in the mid-1900's. Contemporary wind codes are based on a building standard, which is more specific than the early equations such as the one proposed by Stanton. The current building standard was originally published in 1945 by the American National Standards Institute (ANSI), and was intermittently revised. In 1985, the ANSI standard transferred into the American Society of Civil Engineers (ASCE), and the 1988 edition of the building standard became ASCE 7-88. (Liu 1991)

Since 1988, the standard has been significantly refined due to advancing knowledge about how wind affects structures. The advancements are specifically due to new technologies and increased accuracy in predicting wind loads. With the improved methods comes an opportunity to design structures with greater accuracy and without excessive conservatism; however, the methods have also become significantly more complex. In thirty years the wind loads section in building codes went from one page to one hundred, and from one method to three different options. (Taranath 2005) The complexity of the methods has negative implications for design engineers since they have to learn new methods as the code evolves.

The most recent version of the standard, ASCE 7-05, is the version upon which the 2006 IBC and 2008 CBC are based.

2.3 Evolution of Building Codes: The Road to the IBC

The federal government does not regulate building codes in the United States. Instead, individual state and local governments control building regulations. Prior to the late 1990's, there were an estimated 500 different building codes in the United States, but most of them were based on one of three codes: the Uniform Building code, the Standard Building Code, and the Basic Building Code. Until recently, most cities or states adopted one of these three major codes and modified some details in order to suit local requirements. By the end of the 20th century, the Uniform Building Code (UBC) was the most popular among state and city governments. (Liu 1991)

Through the late 1990's the Uniform Building Code was in use in many states. Before the turn of the 21st century, the three major building code organizations, the International Conference of Building Officials, the Southern Building Code Congress International and the Building Official Code Administrators, decided to collaborate on one universal code. This was the beginning of the International Building Code. (IBC) The last UBC was the 1997 edition, and the first IBC was published in the year 2000. The IBC was originally developed by combining the codes from the three organizations. (Hooper 1998)

According to the International Code Council, as of 2006 48 of the 50 states have adopted the IBC in some way (ICC). Some only use the IBC at the local level, but for most states the IBC is effective statewide. The two states that have not yet adopted the IBC are Hawaii and California. The delay in these states, or at least in California, can likely be attributed to more stringent seismic codes in place in these states.

California will adopt the 2006 IBC, with it taking full effect in January of 2008. The result of this building code change is that the CBC will adopt the building section, among other sections, of the IBC. The seismic section will maintain the IBC procedure, but will abandon the seismic zoning system in favor of ground motion contour maps. These contour maps will be more detailed than those that previously existed, especially the ones for the west coast. For the wind section, the CBC will discard its simplified procedure and adopt the more complicated one in the IBC.

3 Impacts of Seismic Code Changes

Seismic and wind loads act in many directions, but are the primary considerations when designing the lateral force resisting system in a structure. It follows that when these design procedures are altered there are impacts on the same elements of the structure. While the affects are not identical, there are similar ramifications for code changes involving either wind or seismic loading. Since similar studies of wind code changes are not as well documented the following sections present case studies of areas that endured a seismic code change. The case studies detail the effects of such changes.

3.1 General Impacts

Half of the world's population lives in major cities. As the urban areas of the world rapidly grow, the devastation due to natural disasters, including earthquakes, becomes of greater concern (Tucker, et al 1994). In addition, as structures become lighter, taller and more structurally efficient, wind loading becomes more and more important. These issues demonstrate the need for building codes to be kept current.

Building codes should keep up with current advancements achieved in research, but how often, and for what reasons, should the code be amended? Code changes usually result from lessons learned from major disasters or from the introduction of new or revised methodologies. According to Sashi Kunnath, a professor at the University of California, Davis, "The first question that comes to mind when introducing a new methodology is the obvious one: why do we need a new procedure? What is inherently wrong or inadequate in the existing provisions for design that warrants a new look at the entire process?" This hurdle is faced with every code revision.

Before the new method of ground motions, a common seismic code change included the revision of the seismic zone for the given area. Revising the zone directly impacts the calculated lateral resistance of a structure (Biggs, et al 1973). Increasing (or decreasing) the seismic zone may improve the theoretical lateral resistance of a structure during a seismic event, but the same change may have other effects, some of which are not favorable.

There are numerous impacts of changing or adopting new building codes. The first is that the lack of continuity may be confusing to the designers and contractors, since having to revise methods on a regular basis is not an ideal situation. When portions of Washington and Oregon adopted the 1994 Uniform Building Code, a publication detailing the changes was distributed, and a series of lectures were offered to the building community to help smooth the transition. This sort of process cannot be carried out every time the code changes. Also, if a code revision comes in the middle of a project, parts of the project may have to be redesigned. This would impact both the cost and timeline of the project. Next, enforcement issues will inevitably arise with frequently revised codes. Finally, after a revision, existing buildings that were originally designed and built correctly are suddenly no longer up to code. Conventional methods of seismic retrofit usually include adding walls, strengthening frames, and other costly methods. Structures that date back to previous, perhaps inadequate, building codes could potentially be very dangerous before they are retrofitted.

3.2 Case Studies

In order to accurately predict effects of upcoming building code changes, it is important to consider what effects previous changes had. Looking to the past is often an effective way to gain insight into what might happen in the future. The following case studies look at the affects of seismic code changes in the past.

3.2.1 **Massachusetts**

Before 1970, Massachusetts had little or no seismic provisions. After some research, the 1970 Uniform Building Code (UBC) implemented a seismic zoning map that put areas of eastern Massachusetts, including Boston and Charleston, in zone 3 (zones range from 0 to 4), and other areas in zones 1 and 2. (Krimgold 1977)

The effect of going from what was either a non-extent or minimal seismic code throughout most of Massachusetts to a relatively comprehensive code resulted in very few problems, but no formal studies were done. Transition to the new code was aided by a comprehensive series of lectures, and Fredrick Krimgold of the Massachusetts Institute of Technology claims "the change caused minimal confusion, and there was not extensive negative reaction concerning the economic impact of the seismic provisions." (Krimgold 1977)

While the confusion and upset was small, the increase in seismic provisions was not without effect. The provisions in the seismic section of the 1970 UBC increased the total cost of construction of steel structures and reinforced concrete structures by 3 percent and 5.5 percent, respectively. Non-structural seismic provisions such as HVAC and ceilings increased total construction cost by about two percent. These increases were based on a single discrete increase in seismic zone. (Leslie, et al 1972)

There seems to be a general lack of awareness about how much a seismic zone increase can actually affect the cost of construction. A five percent cost increase is actually rather substantial. Most of these cost increases are centered on the structural system. It should be noted that these numbers are for the 1970's. The costs for certain materials, such as steel, may have disproportionately increased as compared with other construction costs. Also, the analysis was done using a building with relatively low weight per square foot. The cost increase would be greater for heavier structures. (Leslie, et al 1972)

One positive effect of the seismic zone increase, for designers, is that the number of structures requiring some level of engineering increased drastically, so the previously depressed building industry benefited greatly. This is likely why there were no significant oppositions of the change on the part of the building industry. On the other hand, the lack of opposition could also have been a signal that the new codes were being ignored. (Krimgold 1977)

3.2.2 South Carolina

South Carolina adopted the 2003 International Building Code (IBC) in 2005. This shift in code caused significant changes in seismic requirements that greatly affected building in the area. One specific example of the impacts is a low rise building in Mount Pleasant, South Carolina. Under the IBC a small, 5,300 square-foot building required a far more extensive foundation than under the previous code. Instead of a typical spread footing, deep piles or a floating slab foundation had to be used. In addition, the new code significantly increased the amount of bracing required in the upper portion of the building as well as for ceiling tiles and heating/air conditioning units. The code changes led to more work in all aspects of the project including design, contract documents and construction. Finally, under the revised code, the owner has to pay an independent building inspector to check the structure for code compliance.

Overall, the additional seismic requirements specified in the IBC increased the cost of the building by about five dollars per square foot. Therefore, according to the Charleston Regional Business Journal, "changes to building codes can result in stronger structures, but they can also increase the bottom line and lengthen a project's timeline." (Fisher 2006)

3.2.3 Utah

In 1991, the Uniform Building Code Commission of the State of Utah issued a submittal to the International Conference of Building Officials (ICBO) to change the UBC seismic zone of the Wasatch Front of Utah from zone 3 to zone 4. The petition was denied in 1992 due to insufficient data to support such a change.

A comprehensive analysis of the socio-economic impacts of proposed code changes was conducted (Reavely et al 1993). The socio-economic impacts can be separated into three categories: objective seismic risk, perceived seismic risk, and building code zone. Objective seismic risk is based on real physical data from geological, seismological, geotechnical and engineering sources. Negative impacts of objective seismic risk include increased building costs, and risk-based decisions by people to locate, or relocate, in certain areas. Perceived seismic risk is simply the attitude of citizens toward seismic risk. Fear of earthquakes certainly impacts decisions, but the impact may be less than for other natural disasters. Also, this fear might have positive effects on education, preparedness and risk reduction procedures. Finally, the effects of building code seismic zone seem to be relatively low. A positive effect is the overall better performance of buildings built to code, and the knowledge that a building is built correctly can mitigate the negative effects of objective and perceived seismic risks. There are significant negative effects if the seismic zone is either too high or too low. If it is too high, then the benefits of a better performing building will not validate the costs, and if the zone is too low, the savings from a lower performing building will not equal the costs of the greater damage, losses and even casualties that will inevitably result from a future earthquake. (Reavely, et al 1993)

Fortunately for this area, there are few design-based differences between UBC seismic zone 3 and zone 4, so effects of this change would not significantly affect the design process. Since there are some additional factors to consider when designing for zone 4, the change may make the designing more challenging, but the overall design process will not change, so no additional knowledge on the part of the designer would be required.

It was predicted that the overall impacts on building costs would only be an increase of around one percent since the code change would only significantly affect the design of the structural system, and structural systems make up only about 20 percent of building costs. Renovation and rehabilitation will not be a factor since it will not be required of existing buildings. However, if a building is modified or if its use is changed, seismic retrofit to bring the structure to code would likely be required, and this can be a very expensive process, costing up to around 30 dollars per square foot. (Reavely, et al 1993)

"The impact of a seismic zone change from zone 3 to zone 4 on the value of existing buildings is probably negligible" (Reavely, et al 1993). This is based on the lack of knowledge of difficulty in resale of structures that were built before a code change. This is hard to believe since a buyer may plan to remodel the building, thus having to bring the building to code. This would affect the sale price, thus, the aforementioned prediction is probably not entirely correct.

Finally, costs of insurance would increase due to an increase in seismic zone. On average, masonry buildings in zone 4 have insurance rates 5.6 times greater than those in zone 3, and rates for zone 4 wood frame buildings are about 1.7 times greater than those in Zone 3.

Overall, the impacts of a seismic code change in this area of Utah would be small, and most negative impacts would be due to objective or perceived risk rather than the actual change. It is not surprising that this is the conclusion since the author of the study clearly advocates the change. While it might be a warranted code change, the negative impacts were not addressed candidly. (Reavely, et al 1993)

3.2.4 **Washington**

In 1998 Washington adopted the 1997 Uniform Building Code (UBC). That code change significantly affected the seismic design and constructions of buildings in Seattle and in other areas along the West Coast. If the buildings in these areas are initially designed correctly to code, there should not have been impacts on the project cost. However, retrofit of buildings that are not to code will be incredibly costly. The major changes associated with this code shift included an increased range of soil categories, a new redundancy/reliability factor, details of how elements of a building are tied together structurally and a change to the definition of drift. These changes drastically affected the design and construction process, which eventually results in an impact on the total cost of the structure. (Hooper 1998)

3.2.5 **Summary of Case Studies**

These studies show that changes in seismic code have effects on many aspects of the design and construction of structures. When codes become more stringent, as in the cases of Massachusetts and Utah, the designs of structures become more detailed and costs of materials and construction can increase significantly. When an entirely new code is adopted, as in South Carolina and Washington, the affects can be more widespread. In addition to the increased costs of design and construction, design engineers are required to learn the procedures in the new code, which can be time consuming and inefficient. The learning of the new code is a temporary effect, but increased costs are not. Overall, the aforementioned case studies show that, while code updates are necessary to accurately design structures, the changes do not come without cost.

4 Design Procedures in Current Codes

The following sections will detail the design steps and equations that are specified in each of the codes in question. All equations, figures and tables in the following sections come directly from the codes in question, but they have been organized in a logical manner that shows the steps of the design process.

4.1 2001 California Building Code

4.1.1 Seismic

The purpose and general note about seismic design (Sections 1626.1 and 1626.2) in the 2001 CBC reads as follows:

> The purpose of the earthquake provisions herein is primarily to safeguard against major structural failures and loss of life, not to limit damage or maintain function.
>
> Structures and portions thereof shall, as a minimum, be designed and constructed to resist the effects of seismic ground motions as provided in this division.

The CBC design for earthquake loads considers the following main factors: seismic zones, site characteristics, occupancy and structural system. Seismic zones range from 0 to 4, and a high zone category indicates both a high probability of an earthquake in general, and also a better chance that the earthquake will be of higher magnitude. Zones are determined from a map that can be seen in Figure 4-1.

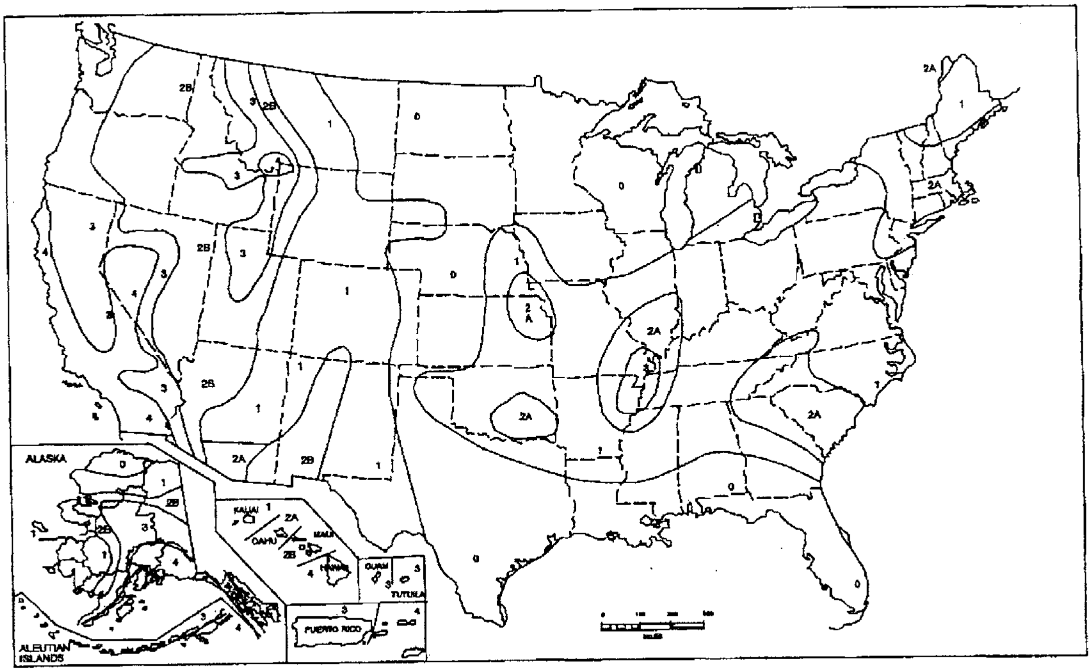


Figure 4-1: Seismic Zoning Map (Equation 16-2 in 2001 CBC)

Once the seismic zone is determined, a seismic zone factor, Z, is assigned according to Table 4-1. The design procedure for zone 4 is more intricate than the procedure for the other zones, as will be described in the following sections.

Table 4-1: Seismic Zone Factor, Z (Table 16-I in 2001 CBC)

Zone	1	2A	2B	3	4
Z	0.075	0.15	0.20	0.30	0.40

Site characteristics refer to details about the soil and other building site factors. A soil profile type is assigned to a site according to Table 4-2.

Table 4-2: Soil Profile Types (Table 16-J in 2001 CBC)

Soil Profile Type	Soil Profile in Name/Generic Description	Average Soil Properties for Top 100ft of Soil Profile		
		Shear Wave Velocity, V_s, in ft/s (m/s)	Standard penetration Test, N, in blows/ft	Undrained Shear Strength, s_u, in psf (kPa)
S_A	Hard Rock	> 5,000 (1,500)		
S_B	Rock	2,500 to 5,000 (760 to 1,500)		
S_C	Very Dense Soil and Soft Rock	1,200 to 2,500 (360 to 760)	> 50	> 2,000 (100)
S_D	Stiff Soil Profile	600 to 1,200 (180 to 360)	15 to 50	1,000 to 2,000 (50 to 100)
S_E	Soft Soil Profile	< 600 (180)	< 15	< 1,000 (50)
S_F	Soil Requiring Site-specific Evaluation			

Occupancy is the use for which the structure is intended; a school or hospital will be subjected to a more rigorous design than an office building. A less detailed version of the occupancy category table is shown in Table 4-3.

Table 4-3: Occupancy Category (from Table 16-K in 2001 CBC)

Occupancy Category	Seismic Importance Factor, I	Wind Importance Factor, I_W
1. Essential Facilities	1.25	1.15
2. Hazardous Facilities	1.35	1.15
3. Special Occupancy Structures	1.00	1.00
4. Standard occupancy Structures	1.00	1.00
5. Miscellaneous Structures	1.00	1.00

Finally, the structural system intended to resist lateral loads is quantified into a factor used in the equations. Table 16-N in the 2001 CBC, which can be found in part in Table 4-4, lists numerous variations of lateral force resisting systems and the corresponding structural system factor, R.

Table 4-4: Response Modification Factor (from Table 16-N in 2001 CBC)

Basic Structural System	Lateral-Force Resisting System Description	R
1. Bearing Wall System	1. 1. Light-framed walls with shear panels	
	a. Wood structural panel walls for structures three stories or less	5.5
	b. All other light-framed walls	4.5
	2. Shear walls	
	a. Concrete	4.5
	b. Masonry	4.5
	3. Light steel-framed bearing walls with tension-only bracing	2.8
	4. Braced frames where bracing carries gravity load	
	a. Steel	4.4
	b. Concrete	2.8
	c. Heavy Timber	2.8
2. Building Frame System	1. Steel eccentrically braced frame	7.0
	2. Light-framed walls with shear panels	
	a. Wood structural panel walls for structures three stories or less	6.5
	b. All other light-framed walls	5.0
	3. Shear Walls	
	a. Concrete	5.5
	b. Masonry	5.5
	4. Ordinary braced frames	
	a. Steel	5.6
	b. Concrete	5.6
	c. Heavy Timber	5.6
	5. Special concentrically braced frames	
	a. Steel	6.4
3. Moment-resisting Frame System	1. Special moment-resisting frame	
	a. Steel	8.5
	b. Concrete	8.5
	2. Masonry moment-resisting wall frame	6.5
	3. Concrete intermediate moment-resisting frame	5.5
	4. Ordinary moment-resisting frame	
	a. Steel	4.5
	b. Concrete	3.5
	5. Special truss moment frames of steel	6.5

Seismic design is derived from the base shear, V, produced by an earthquake. Base shear is determined by the following equations. Note that the minimum base shear equation for zone 4 differs from the minimum for the other zones.

$$V = \frac{C_v I}{RT} W$$

Equation 4-1: Base Shear (Equation 30-4 in 2001 CBC)

$$V = \frac{2.5 C_a I}{R} W$$

Equation 4-2: Maximum Base Shear (Equation 30-5 in 2001 CBC)

$$V = 0.11 C_a I W$$

Equation 4-3: Minimum Base Shear (Equation 30-6 in 2001 CBC)

$$V = \frac{0.8 Z N_v I}{R} W$$

Equation 4-4: Zone 4 Minimum Base Shear (Equation 30-7 in 2001 CBC)

Where:
V is the design base shear in pounds
C_v is a seismic coefficient that can be found in Table 4-5, below
C_a is a seismic coefficient that can be found in Table 4-6, below
I is the seismic importance factor
R is the structural system factor
T is the period of the structure, in seconds, as calculated in Equation 4-5
W is the total mass of the structure, in pounds
N_v is a near-source factor that can be found in Table 4-7
N_a is a near-source factor that can be found in Table 4-8

Table 4-5: Seismic Coefficient C_v (Table 16-R in 2001 CBC)

	Seismic Zone Factor, Z				
Soil Profile Type	Z = 0.075	Z = 0.15	Z = 0.20	Z = 0.30	Z = 0.40
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$
S_F	Requires Site-specific Evaluation				

Table 4-6: Seismic Coefficient C_a (Table 16-Q in 2001 CBC)

	Seismic Zone Factor, Z				
Soil Profile Type	Z = 0.075	Z = 0.15	Z = 0.20	Z = 0.30	Z = 0.40
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
S_F	Requires Site-specific Evaluation				

Table 4-7: Near-Source Factor N_v (Table 16-T in 2001 CBC)

	Closest Distance to Known Seismic Source		
Seismic Source Type	≤ 2 km	5 km	≥10 km
A	2.0	1.6	1.2
B	1.6	1.2	1.0
C	1.0	1.0	1.0

Table 4-8: Near-Source Factor N_a (Table 16-S in 2001 CBC)

	Closest Distance to Known Seismic Source		
Seismic Source Type	≤ 2 km	5 km	≥10 km
A	1.5	1.2	1.0
B	1.3	1.0	1.0
C	1.0	1.0	1.0

The seismic source type depends on fault types and maximum magnitudes. Table 4-9 shows how to determine the seismic source type.

Table 4-9: Seismic Source Type (Table 16-U in 2001 CBC)

Seismic Source Type	Seismic Source Description	Seismic Source Definition: Maximum Moment Magnitude, M	Seismic Source Definition: Slip Rate, SR (mm/year)
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity	M ≥7.0	SR ≥5
B	All faults other than Types A and C	M ≥7.0 M <7.0 M ≥6.5	SR >5 SR >2 SR <2
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity	M<6.5	SR≤2

The period of a building may be estimated in two ways, method A is as follows:

$$T = C_t(h_n)^{3/4}$$

Equation 4-5: Period (Equation 30-8 in 2001 CBC)

Where: T is the natural period of the structure, in seconds
C_t = 0.035 for steel moment-resisting frames
C_t = 0.030 for reinforced concrete moment-resisting frames and eccentrically braced frames
C_t = 0.020 for all other cases
h_n = height of structure to the level n, in feet

Once the base shear is determined, it is distributed up the building in a triangular fashion and the lateral resisting system is designed accordingly.

4.1.2 Wind

The method for designing a structure to resist wind loads in the CBC is based on the ASCE 7-88 Building Standard, but with some simplifying assumptions to make the calculations easier. This simplified procedure, which is reasonable since wind rarely controls in California, is only one equation. Equation 4-6 calculates the design wind pressure necessary to design a structure.

$$P = C_e C_q q_s I_w$$

Equation 4-6: Wind Pressure (Equation 20-1 in 2001 CBC)

where: P is the design wind pressure, in psf
C_e is the combined height, exposure and gust factor shown in Table 4-11
C_q is the pressure coefficient for the structure or the portion of the structure under consideration. The table for this coefficient can be found in Table 4-10
q_s is the wind stagnation pressure, in psf, at the standard height of 33 feet, shown in Table 4-12
I_w is the wind importance factor, which can be seen in Table 4-3

Table 4-10: Pressure Coefficient (Table 16-H in 2001 CBC)

STRUCTURE OR PART THEREOF	DESCRIPTION	C_q FACTOR
1. Primary frames and systems	Method 1 (Normal force method) Walls: Windward wall Leeward wall Roofs[1]: Wind perpendicular to ridge Leeward roof or flat roof Windward roof less than 2:12 (16.7%) Slope 2:12 (16.7%) to less than 9:12 (75%) Slope 9:12 (75%) to 12:12 (100%) Slope > 12:12 (100%) Wind parallel to ridge and flat roofs	 0.8 inward 0.5 outward 0.7 outward 0.7 outward 0.9 outward or 0.3 inward 0.4 inward 0.7 inward 0.7 outward
	Method 2 (Projected area method) On vertical projected area Structures 40 feet (12 192 mm) or less in height Structures over 40 feet (12 192 mm) in height On horizontal projected area[1]	 1.3 horizontal any direction 1.4 horizontal any direction 0.7 upward
2. Elements and components not in areas of discontinuity[2]	Wall elements All structures Enclosed and unenclosed structures Partially enclosed structures Parapets walls	 1.2 inward 1.2 outward 1.6 outward 1.3 inward or outward
	Roof elements[3] Enclosed and unenclosed structures Slope < 7:12 (58.3%) Slope 7:12 (58.3%) to 12:12 (100%) Partially enclosed structures Slope < 2:12 (16.7%) Slope 2:12 (16.7%) to 7:12 (58.3%) Slope > 7:12 (58.3%) to 12:12 (100%)	 1.3 outward 1.3 outward or inward 1.7 outward 1.6 outward or 0.8 inward 1.7 outward or inward
3. Elements and components in areas of discontinuities[2,4,5]	Wall corners[6] Roof eaves, rakes or ridges without overhangs[6] Slope < 2:12 (16.7%) Slope 2:12 (16.7%) to 7:12 (58.3%) Slope > 7:12 (58.3%) to 12:12 (100%) For slopes less than 2:12 (16.7%) Overhangs at roof eaves, rakes or ridges, and canopies	1.5 outward or 1.2 inward 2.3 upward 2.6 outward 1.6 outward 0.5 added to values above
4. Chimneys, tanks and solid towers	Square or rectangular Hexagonal or octagonal Round or elliptical	1.4 any direction 1.1 any direction 0.8 any direction
5. Open-frame towers[7,8]	Square and rectangular Diagonal Normal Triangular	 4.0 3.6 3.2
6. Tower accessories (such as ladders, conduit, lights and elevators)	Cylindrical members 2 inches (51 mm) or less in diameter Over 2 inches (51 mm) in diameter Flat or angular members	 1.0 0.8 1.3
7. Signs, flagpoles, lightpoles, minor structures[8]		1.4 any direction

[1]For one story or the top story of multistory partially enclosed structures, an additional value of 0.5 shall be added to the outward C_q. The most critical combination shall be used for design. For definition of partially enclosed structures, see Section 1616.

[2]C_q values listed are for 10-square-foot (0.93 m^2) tributary areas. For tributary areas of 100 square feet (9.29 m^2), the value of 0.3 may be subtracted from C_q, except for areas at discontinuities with slopes less than 7 units vertical in 12 units horizontal (58.3% slope) where the value of 0.8 may be subtracted from C_q. Interpolation may be used for tributary areas between 10 and 100 square feet (0.93 m^2 and 9.29 m^2). For tributary areas greater than 1,000 square feet (92.9 m^2), use primary frame values.

[3]For slopes greater than 12 units vertical in 12 units horizontal (100% slope), use wall element values.

[4]Local pressures shall apply over a distance from the discontinuity of 10 feet (3048 mm) or 0.1 times the least width of the structure, whichever is smaller.

[5]Discontinuities at wall corners or roof ridges are defined as discontinuous breaks in the surface where the included interior angle measures 170 degrees or less.

[6]Load is to be applied on either side of discontinuity but not simultaneously on both sides.

[7]Wind pressures shall be applied to the total normal projected area of all elements on one face. The forces shall be assumed to act parallel to the wind direction.

[8]Factors for cylindrical elements are two thirds of those for flat or angular elements.

Table 4-11: Combined Height, Exposure, Gust Factor (Table 16-G in 2001 CBC)*

Height above average level of adjoining ground, ft	Exposure D	Exposure C	Exposure B
0-15	1.39	1.06	0.62
20	1.45	1.13	0.67
25	1.50	1.19	0.72
30	1.54	1.23	0.76
40	1.62	1.31	0.84
60	1.73	1.43	0.95
80	1.81	1.53	1.04
100	1.88	1.61	1.13
120	1.93	1.67	1.20
160	2.02	1.79	1.31
200	2.10	1.87	1.42
300	2.23	2.05	1.63
400	2.34	2.19	1.80

*Values for intermediate heights above 15 feet may be interpolated.

Where Exposure B "has terrain with buildings, forest or surface irregularities, covering at least 20 percent of the ground level area extending 1 mile or more from the site," Exposure C "has terrain that is flat and generally open, extending a half mile or more from the site in any full quadrant," and Exposure D "represents the most severe exposure in areas with basic wind speeds of 80 miles per hour or greater and has terrain that is flat and unobstructed facing large bodies of water over one mile or more in width relative to any shoreline quarter mile or 10 times the building height, whichever is greater".

Table 4-12: Wind Stagnation Pressure (Table 16-F in 2001 CBC)

Basic wind speed, mph	70	80	90	100	110	120	130
Pressure, q_s, psf	12.6	16.4	20.8	25.6	31.0	36.9	43.3

The basic wind speeds can be found on a map shown in Figure 4-2.

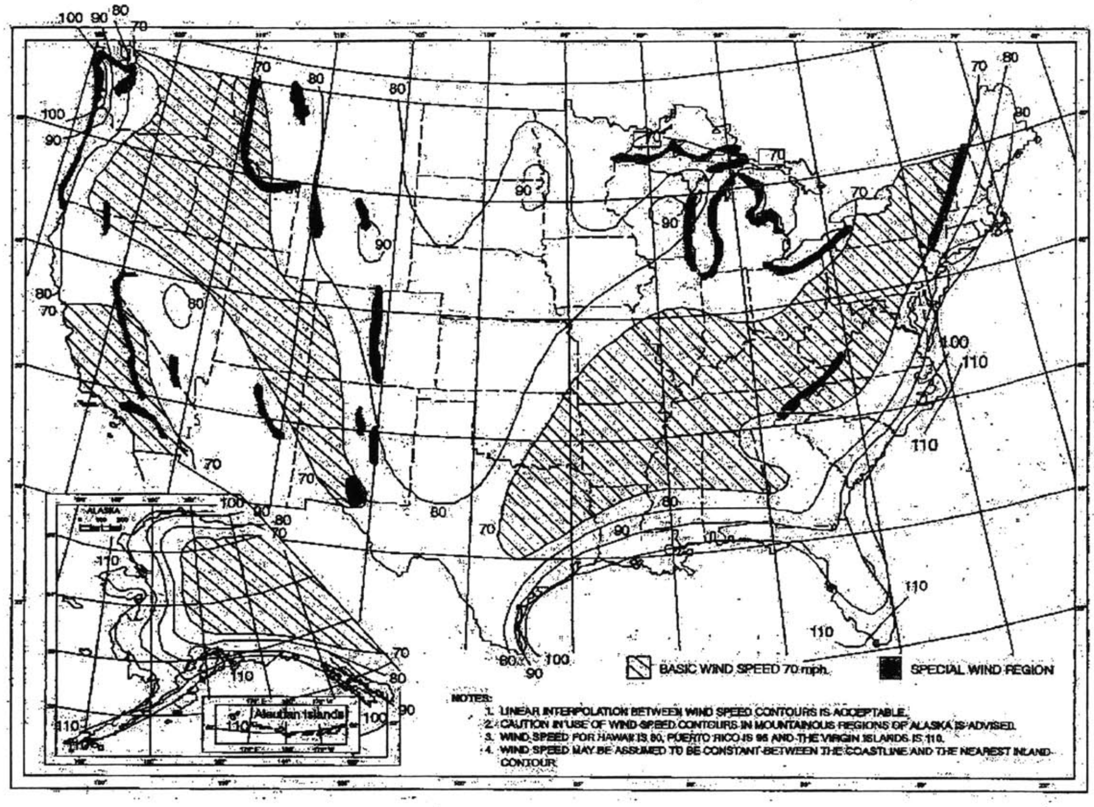


Figure 4-2: Minimum Wind Speed (Figure 16-1 in 2001 CBC)

The pressure resulting from Equation 4-6 can be converted into an equivalent base shear by modeling the building as a cantilever beam and calculating shear force at the base of the building. This base shear would then be compared to the seismic base shear; the larger value would be the design value.

4.2 2003 International Building Code

Until January 1, 2007 most of the United States depended on the 2003 IBC. The following describes the methods for determining both the wind and seismic loads on structures.

4.2.1 Seismic

The first part of the scope of the section on earthquake loads (Section 1614) in the 2003 IBC reads as follows:

> Every Structure, and portion thereof, shall as a minimum be designed and constructed to resist the effects of earthquake motions and assigned a seismic design category…

While much of the figures, tables and equations are copied into the pages of the code, the IBC follows the American Society of Civil Engineers' Building Standard 7. The newest version of the ASCE 7, the 2005 version, describes seismic loading in terms of base shear. The following equations detail the basic procedure.

$$V = C_S W$$

Equation 4-7: Base Shear (Equation 12.8-1 in ASCE 7-05)

where: V is the design base shear, in pounds
C_s is the Seismic Response coefficient, defined in Equation 4-8, below
W is the effective seismic weight of the structure in pounds

$$C_s = \frac{S_{DS}}{T(\frac{R}{I})}$$

Equation 4-8: Seismic Response Coefficient (Equation 12.8-2 in ASCE 7-05)

$$C_s = \frac{S_{D1}}{T(\frac{R}{I})}$$

Equation 4-9: Cs Maximum (Equation 12.8-3 in ASCE 7-05)

where: S_{DS} is the design spectral response acceleration parameter in the short period range, as later determined by Equation 4-12, in percent of acceleration due to gravity, g
R is the response modification factor, a tabulated value that can be found in Table 4-13
I is the occupancy importance factor from Table 4-14
S_{D1} is the design spectral response acceleration parameter at a period of 1.0 second, as later determined by Equation 4-13, in percent of g
T is the fundamental period of the structure, in seconds

The minimum value for C_s is 0.01. However, if S_1, the mapped maximum considered earthquake spectral response acceleration parameter (in percent of g, the acceleration due to gravity), is greater than 0.6g, then C_s shall not be less than the outcome of the following equation.

$$C_s = \frac{0.5 S_1}{\left(\frac{R}{I}\right)}$$

Equation 4-10: Minimum Cs for S_1 >0.6g (Equation 12.8-6 in ASCE 7-05)

Table 4-13: Response Modification Coefficient (from Table 12.2-1 in ASCE 7-05)

Seismic Force-Resisting System	Response Modification Coefficient, R
BEARING WALL SYSTEMS	
1. Special reinforced concrete shear walls	5
2. Ordinary reinforced concrete shear walls	4
3. Detailed plain concrete shear walls	2
4. Ordinary plain concrete shear walls	1.5
5. Intermediate precast shear walls	4
6. Ordinary precast shear walls	3
7. Special reinforced masonry shear walls	5
8. Intermediate reinforced masonry shear walls	3.5
9. Ordinary reinforced masonry shear walls	2
10. Detailed plain masonry shear walls	2
11. Ordinary plain masonry shear walls	1.5
12. Prestressed masonry shear walls	1.5
13. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	6.5
14. Light-framed walls with shear panels of all other materials	2
15. Light-framed wall systems using flat strap bracing	4
BUILDING FRAME SYSTEMS	
1. Steel eccentrically braced frames, moment resisting connections at columns away from links	8
2. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links	7
3. Special steel concentrically braced frames	6
4. Ordinary steel concentrically braced frames	3.25
5. Special reinforced concrete shear walls	6
6. Ordinary reinforced concrete shear walls	5

7. Detailed plain concrete shear walls	2
8. Ordinary plain concrete shear walls	1.5
9. Intermediate precast shear walls	5
10. Ordinary precast shear walls composite steel and concrete eccentrically braced frames	4
11. Composite steel and concrete eccentrically braced frames	8
12. Composite steel and concrete concentrically braced fames	5
13. Ordinary composite steel and concrete braced fames	3
14. Composite steel plate shear walls	6.5
15. Special composite reinforced concrete shear walls with steel elements	6
16. Ordinary composite reinforced concrete shear walls with steel elements	5
17. Special reinforced masonry shear walls	5.5
18. Intermediate reinforced masonry shear walls	4
19. Ordinary reinforced masonry shear walls	2
20. Detailed plain masonry shear walls	2
21. Ordinary plain masonry shear walls	1.5
22. Prestressed masonry shear walls	1.5
23. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	7
24. Light-framed walls with shear panels of all other materials	2.5
25. Buckling-restrained braced frames, non-moment-resisting beam-column connections	7
26. Buckling-restrained braced frames, moment-resisting beam-column connections	8
27. Special steel plate shear walls	7
MOMENT-RESISTING FRAME SYSTEMS	
1. Special steel moment frames	8
2. Special steel truss moment frames	7
3. Intermediate steel moment frames	4.5
4. Ordinary steel moment frames	3.5
5. Special reinforced concrete moment frames	8
6. Intermediate reinforced concrete moment frames	5
7. Ordinary reinforced concrete moment frames	3
8. Special composite steel and concrete frames	8
9. Intermediate composite moment frames	5
10. Composite partially restrained moment frames	6
11. Ordinary composite moment frames	3

The Response Modification Factor, R, accounts for inelastic qualities of a structure. During an earthquake, structural (and non-structural) elements will yield, increasing the ductility, which causes damping, or absorption of energy. In addition, increased ductility will increase the fundamental period of a structure, which changes the response. The higher values of R signify a greater amount of expected yielding before failure. The design response felt by a structure is dependent on elastic assumptions, but inelasticity occurs during seismic events, so there is a need for the R factor.

Table 4-14: Importance Factor (Table 11.5-1 in ASCE 7-05)

Occupancy Category	I
I or II	1.0
III	1.25
IV	1.5

The occupancy category required to determine the importance factor is based on the intended use of the structure. Occupancy scenarios are detailed and assigned an occupancy category that is used in determining the importance factor. This table can be found in Table 4-15.

Table 4-15: Occupancy Category (from Table 1.1 in ASCE 7-05)

Nature of Occupancy	Occupancy Category
Buildings and other structures that represent a low hazard to human life in the event of failure...	I
All buildings and other structures except those listed in Occupancy Categories I, III and IV	II
Buildings and other structures that represent a substantial hazard to human life in the event of a failure...	III
Buildings and other structures designated as essential facilities...	IV

The fundamental period of the structure, T, may be determined in one of two ways. The first is through using "the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis." Alternately, T may be estimated using the approximate fundamental period, T_a, with Equation 4-11. This value is also the upper limit of any value determined in analysis.

$$T_a = C_t h_n^x$$

Equation 4-11: Fundamental Period (Equation 12.8-7 in ASCE 7-05)

where: T_a is the approximate fundamental period of the structure, in seconds
h_n is the height in feet above the base to the highest level of the structure
C_t and x are coefficients determined from Table 4-16

Table 4-16: Approximate Period Parameters (from Table 12.8-2 in ASCE 7-05)

Structure Type	C_t (ft)	x
Steel moment-resisting frames	0.028	0.8
Concrete moment-resisting frames	0.016	0.9
Eccentrically braced steel frames	0.03	0.75
All other structural systems	0.02	0.75

The design spectral response acceleration parameters, S_{DS} and S_{D1}, used in Equation 4-9 and Equation 4-20, shall be determined by Equation 4-12 and Equation 4-13.

$$S_{DS} = \frac{2}{3} S_{MS}$$

Equation 4-12: 5% Damped Spectral Response Acceleration at Short Periods (Equation 16-40 in 2003 IBC)

$$S_{D1} = \frac{2}{3} S_{M1}$$

Equation 4-13: 5% Damped Spectral Response Acceleration at 1-second Period (Equation 16-41 in 2003 IBC)

Where: S_{DS} and S_{D1} are the design spectral response accelerations in %g
S_{MS} is the maximum considered earthquake spectral response accelerations for short period as found in Equation 4-14 in %g
S_{M1} is the maximum earthquake spectral response acceleration for a 1-second period, as found in Equation 4-15, in %g

$$S_{MS} = F_a S_s$$

Equation 4-14: Spectral Response Acceleration at Short Period (Equation 16-38 in 2003 IBC)

$$S_{M1} = F_v S_1$$

Equation 4-15: Spectral Response Acceleration at 1-second Period (Equation 16-39 in 2003 IBC)

where: F_a is a site coefficient defined in Table 4-17
F_v is a site coefficient defined in Table 4-18
S_s is the mapped spectral acceleration for short periods
S_1 is the mapped spectral acceleration for a 1-second period

Table 4-17: Site Coefficient Fa[a] (Table 1615.1.2(1) in 2003 IBC)

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, S_s
b. Values shall be determined in accordance with section 11.4.7 of ASCE 7

Table 4-18: Site Coefficient F_v[a] (Table 1615.1.2(2) in 2003 IBC)

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_1 \leq 0.25$	$S_1 = 0.50$	$S_1 = 0.75$	$S_1 = 1.00$	$S_1 \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, S_1

b. Values shall be determined in accordance with section 11.4.7 of ASCE 7

The site classes, A through F, are determined by the type of soil that exists at the site, as shown in Table 4-19.

Table 4-19: Site Classes (from section 1615.1.5 in 2003 IBC and Table 1613.5.2 in 2006 IBC)

Site Class	Soil Profile Name
A	Hard rock
B	Rock
C	Very dense soil and soft rock
D	Stiff soil profile
E	Soft soil profile

The mapped spectral accelerations are found on earthquake ground motion contour maps. The values for the accelerations shown on the maps are shown as percentages of acceleration due to gravity, g. Some of these maps can be found in Figure 4-3 (S_S for the western US), Figure 4-4 (S_S for the eastern US), Figure 4-5 (S_1 for the western US) and Figure 4-6 (S_1 for the eastern US).

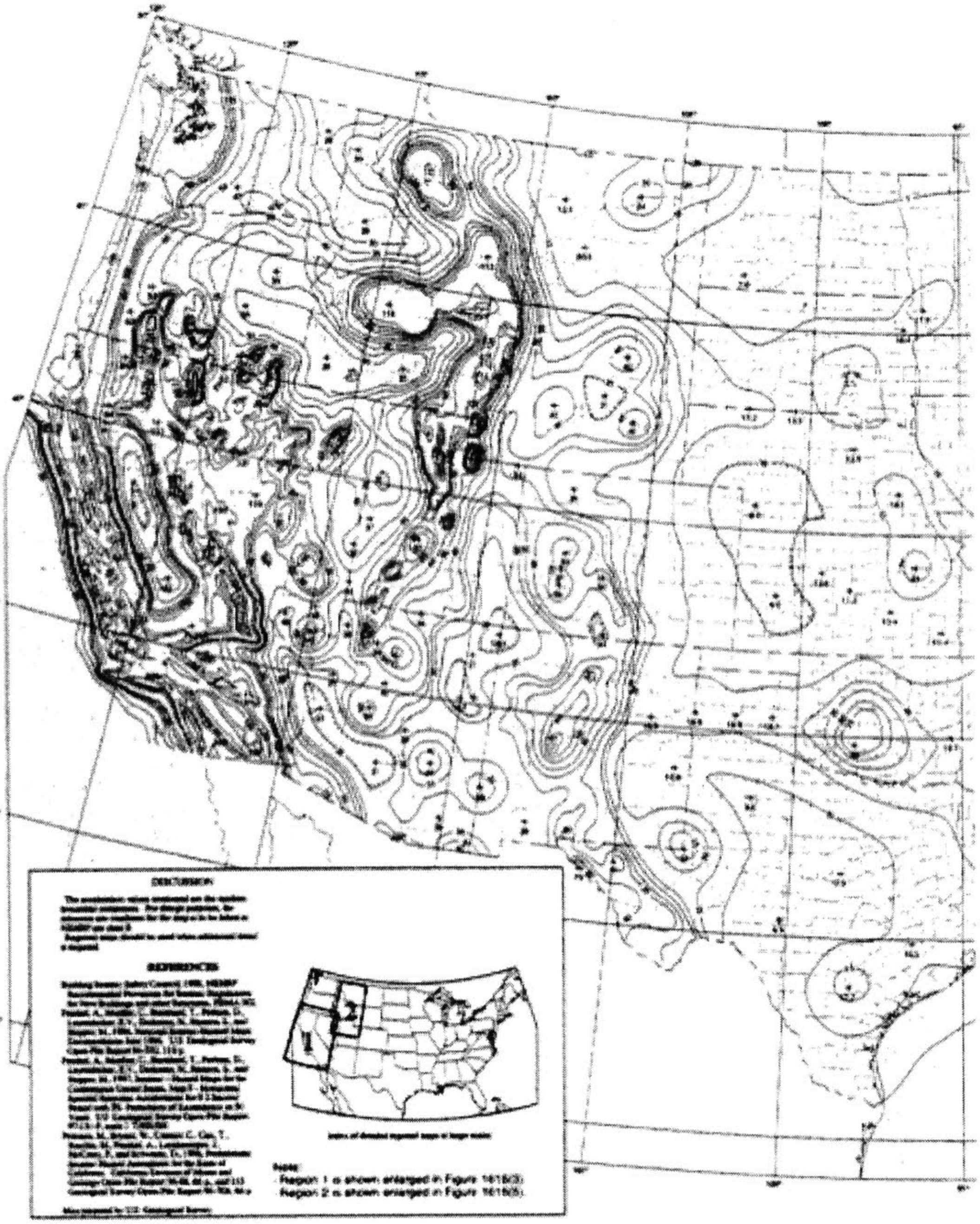


Figure 4-3: Maximum Considered Earthquake Ground Motion for 0.2 Second Spectral Response Acceleration, 5% Damping (Figure 1615(1) in 2003 IBC)

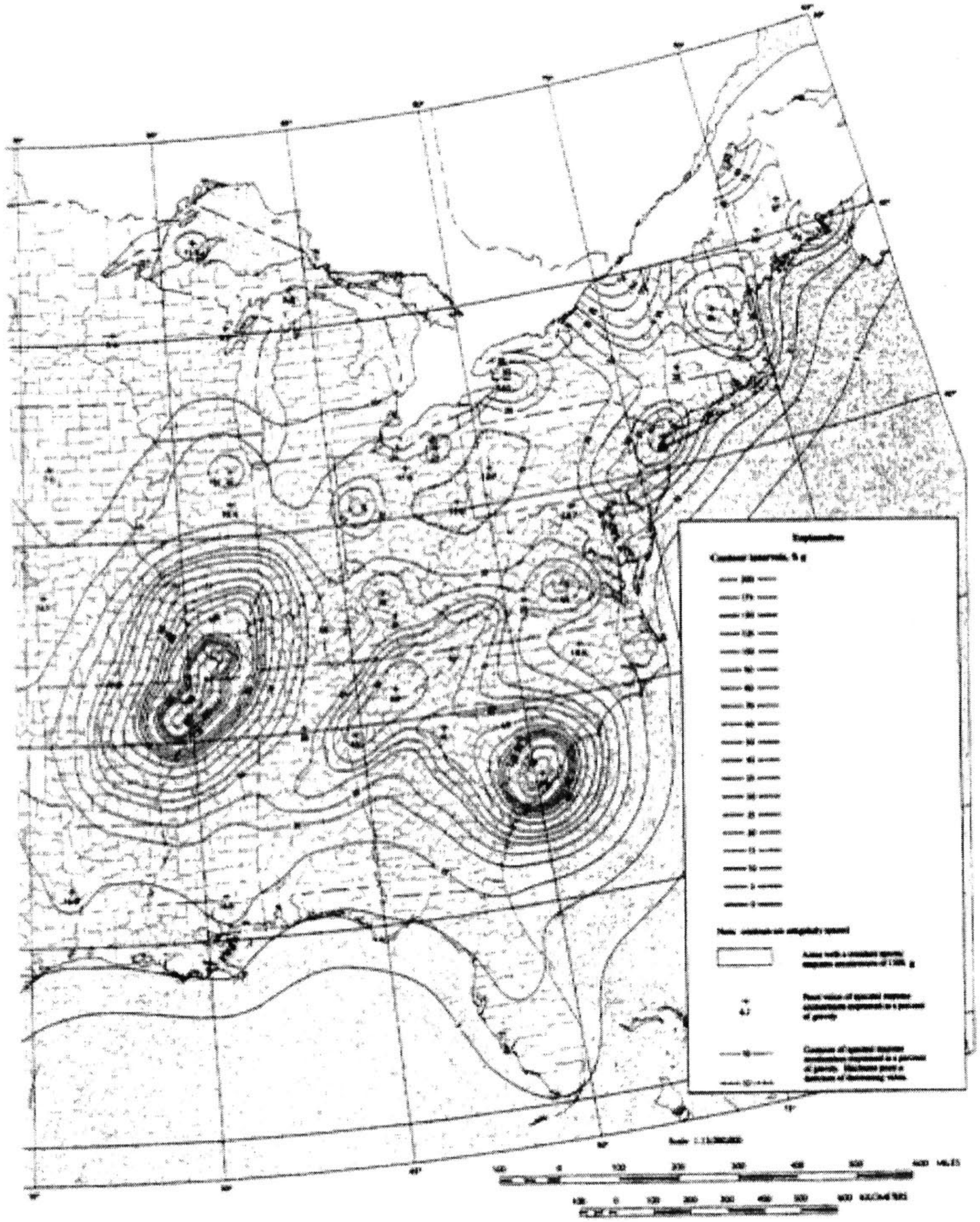


Figure 4-4: Maximum Considered Earthquake Ground Motion for 0.2 Second Spectral Response Acceleration, 5% Damping (Figure 1615(1) in 2003 IBC)

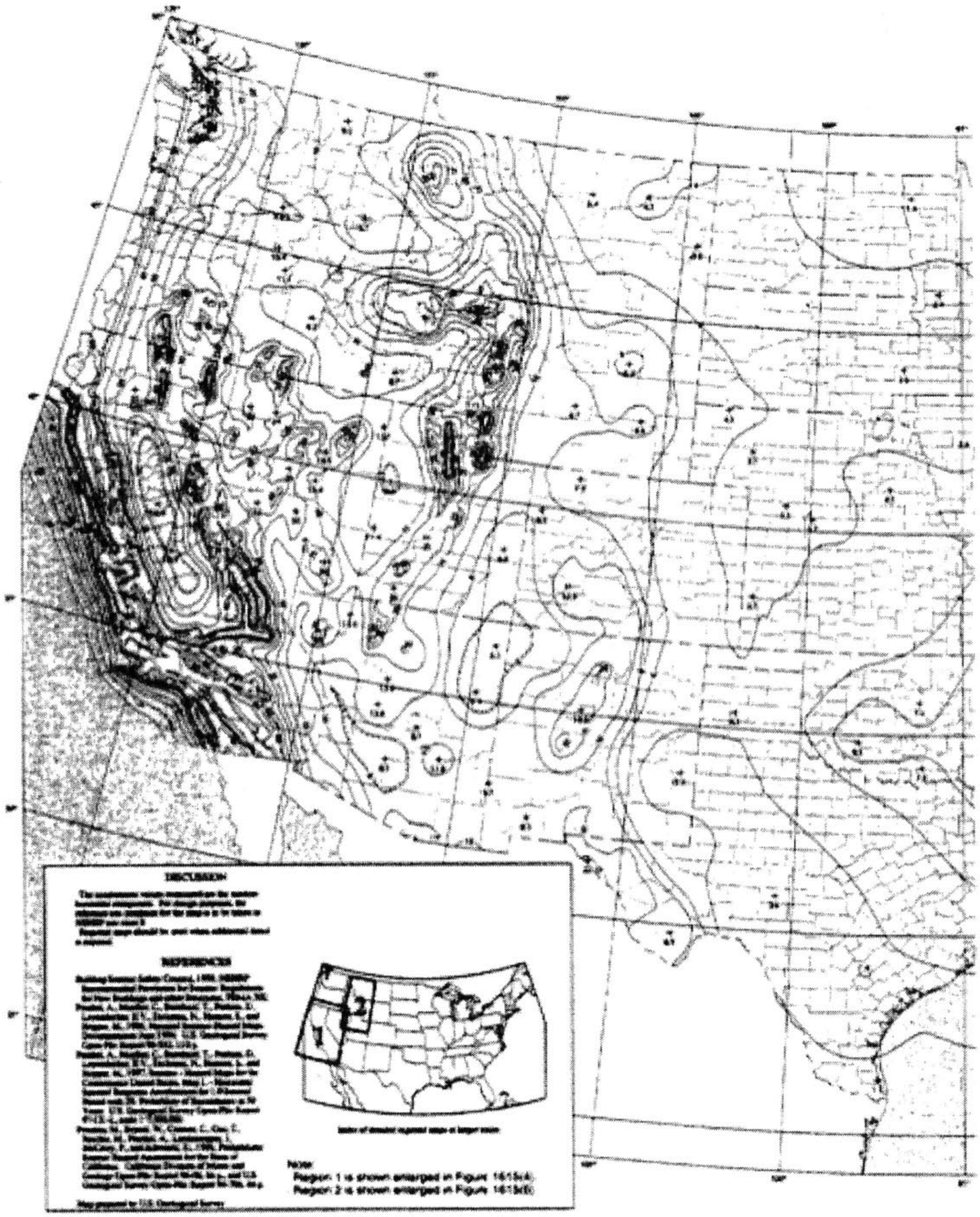


Figure 4-5: Maximum Considered Earthquake Ground Motion for 1.0 Second Spectral Response Acceleration, 5% Damping (Figure 1615(2) in 2003 IBC)

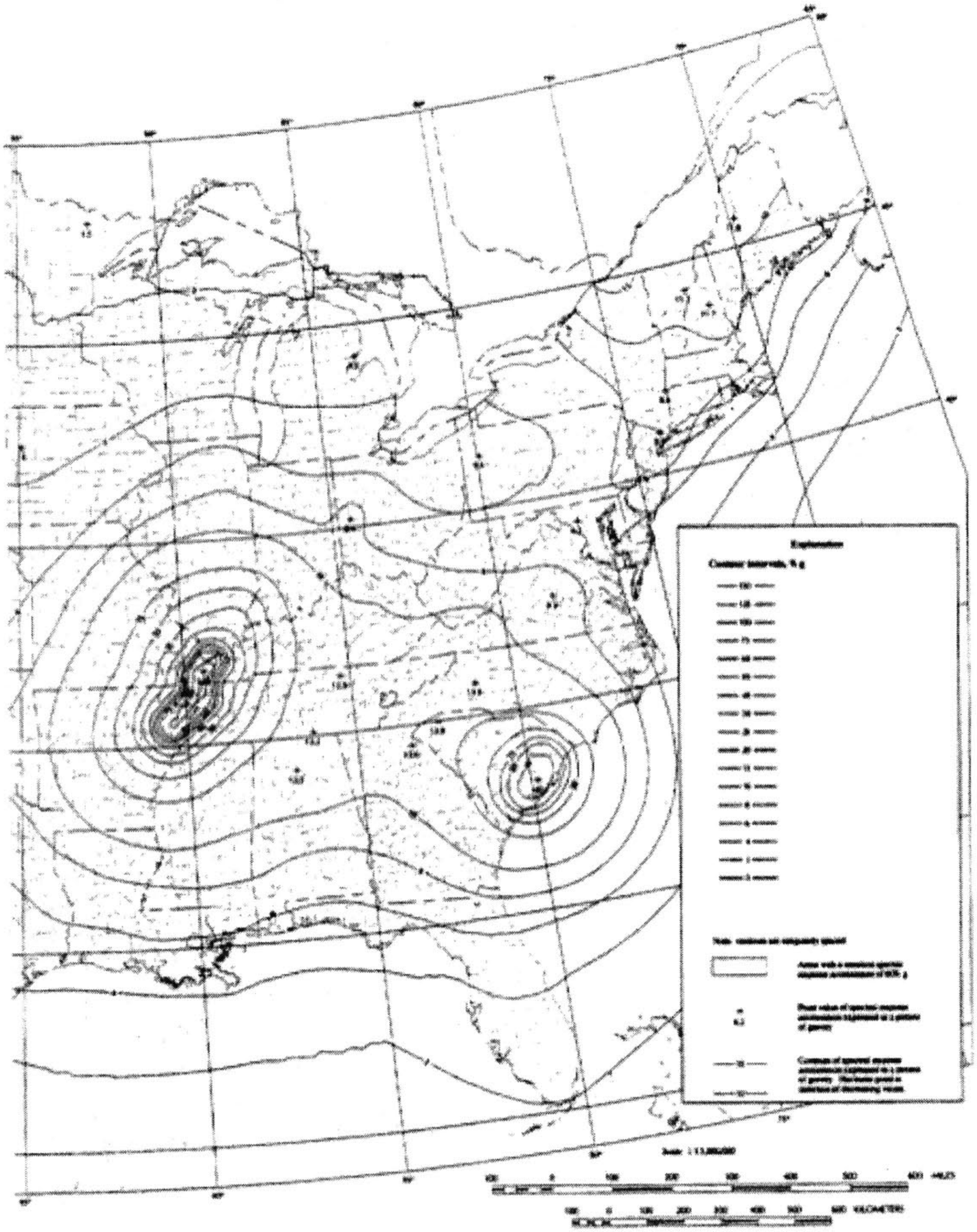


Figure 4-6: Maximum Considered Earthquake Ground Motion for 1.0 Second Spectral Response Acceleration, 5% Damping (Figure 1615(2) in 2003 IBC)

Once S_{DS} and S_{D1} are determined, a design response acceleration spectrum can be constructed. A response spectrum is a plot of the peak, steady-state response acceleration, in percent of the acceleration due to gravity (could also be velocity or displacement), versus the period of the motion, in seconds, of a single degree of freedom oscillator, based on empirical data. The plot creates a way to predict the response of a structure in an earthquake as a function of period. Once the natural frequency and period of a structure are determined, the peak expected response of the building is the corresponding value on the response spectrum. This value is then used to determine the minimum load that the structure must resist. The shape of a response spectrum used in the IBC is more or less a curve-fit of measured acceleration data during an earthquake, and the response at lower periods is greater than that at longer periods. This is shown in Figure 4-7.

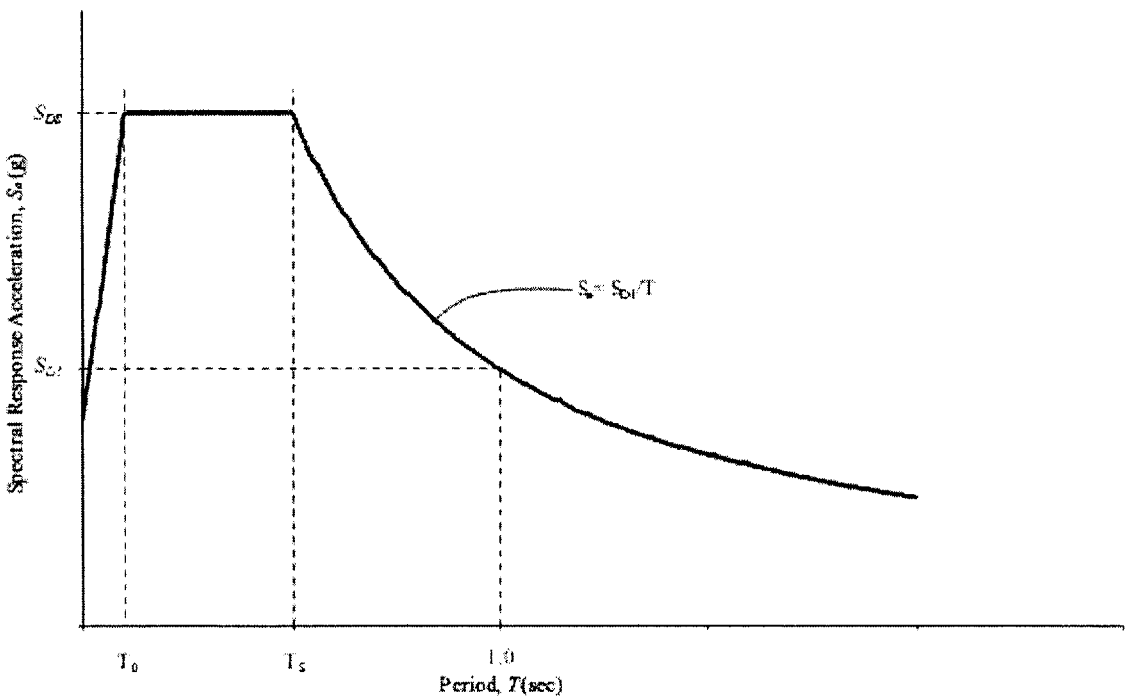


Figure 4-7: Design Response Spectrum (Figure 1615.1.4 in 2003 IBC)

T_0 and T_S (the interval of the fundamental period of the structure within which the spectral response acceleration is constant, and maximized) are defined using Equation

4-16 and Equation 4-17. The section after T_S represents periods that correspond to a response that has constant spectral velocity.

$$T_0 = 0.2\frac{S_{D1}}{S_{DS}}$$

Equation 4-16: T_0 (from section 1615.1.4 in 2003 IBC and 11.4.4 in ASCE 7-05)

$$T_S = \frac{S_{D1}}{S_{DS}}$$

Equation 4-17: T_S (from section 1615.1.4 in 2003 IBC and 11.4.4 in ASCE 7-05)

where: T_0 and T_S are fundamental periods, in seconds

S_{D1} and S_{DS} are spectral acceleration parameters in percent g

The above details all of the basic equations to determine the design base shear values according to the 2003 IBC. From this point, the base shear would be distributed throughout the stories, and the lateral resisting system would be designed accordingly.

4.2.2 Wind

According to Section 1609.1.1 I the 2006 International Building Code, "Wind loads on every building or structure shall be determined in accordance with Chapter 6 of ASCE 7…" Similarly to the seismic section, the wind section of the IBC is based on the ASCE building standard. In ASCE 7, there are three accepted methods for designing the "Main Wind-Force Resisting System," or "MWFRS". The methods are the Simplified Procedure, the Analytical procedure and the Wind Tunnel Procedure, or Method 1, Method 2 and Method 3, respectively. In order for a structure and all its components to be designed using Method 1, there are several conditions the structure must meet. Some of these requirements are that the mean roof height of the structure does not exceed the least horizontal dimension or 60 feet, the structure has to be a regular shape, have a natural frequency less than 1 HZ, and all of the lateral loads have to be transmitted through roof or floor diaphragms into the same MWFRS (no structural inconsistencies).

It is likely that many low-rise, basic structures would qualify to be designed under Method 1, but anything over a 60 feet tall or anything irregular will have to be designed using Method 2. Method 3 will only be necessary in very specific cases.

The design procedure for Method 1 consists of an equation to determine the net wind pressure, shown below in Equation 4-18.

$$p_s = \lambda K_{zt} I p_{S30}$$

Equation 4-18: Wind Net Pressure (Equation 6-1 in ASCE 7-05)

where:
p_s is the net wind pressure in psf
λ is a building height adjustment factor according to Table 4-20
K_{zt} is a topographic factor defined by
I is an importance factor as in Table 4-14
p_{S30} is the simplified design wind pressure, in psf, for Exposure B, at h=30 ft, and for I = 1.0

This equation looks very different from one of the original equations, as shown in Equation 2-4, but uses the same philosophy. In Equation 2-4 the wind pressure is determined using maximum wind velocity and a drag coefficient. Equation 4-18 scales a basic wind pressure value, p_{S30}, which is based on factors such as maximum wind velocity and site conditions. This value is then scaled by the other factors, such as K_{zt}, a topographic factor, which is similar to the drag coefficient factor in the original equation. While this equation has evolved over time, it is still possible to see the connections to the original equations.

Table 4-20: Building Height Adjustment Factor

Mean roof height (ft)	Exposure		
	B	C	D
15	1.00	1.21	1.47
20	1.00	1.29	1.55
25	1.00	1.35	1.61
30	1.00	1.40	1.66
35	1.05	1.45	1.70
40	1.09	1.49	1.74
45	1.12	1.53	1.78
50	1.16	1.56	1.81
55	1.19	1.59	1.84
60	1.22	1.62	1.87

The roughness categories to which the exposure categories refer are detailed in the code. Surface Roughness B applies to suburban and urban areas, Surface Roughness C refers to open terrain, and Surface Roughness D is flat.

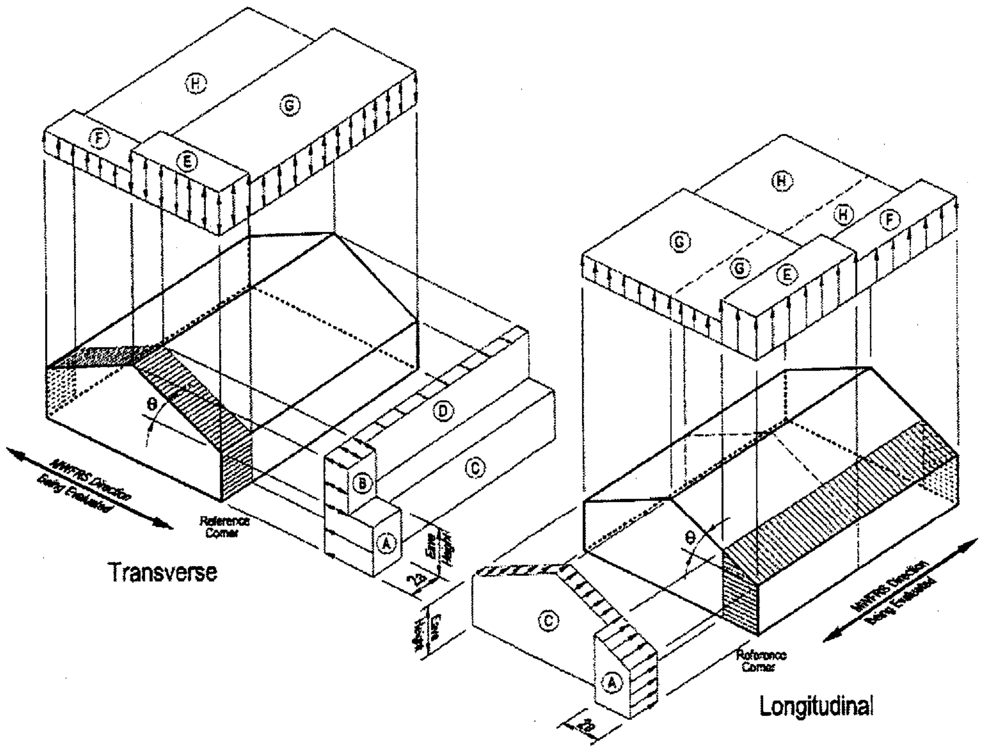
The topographic factor, K_{zt}, is defined by the following equation:

$$K_{zt} = (1 + K_1 K_2 K_3)$$

Equation 4-19: Topographical Factor (Equation 6-3 in ASCE 7-05)

where K_1, K_2 and K_3 are factors determined by the topography around the building site. K_{zt} is tabulated in ASCE 7.

The simplified design wind pressure, p_{S30}, is given in ASCE 7 as a set of tables (partially shown in Table 4-21) based on the area upon which the wind acts and the basic wind speed. The zones (A-H) are shown in Figure 4-8. The basic wind speed is determined using a 3-second gust speed that is a mapped value. The national map is shown in Figure 4-9 (western US and Alaska) and Figure 4-10 (Eastern US).



Notes:

1. Pressures shown are applied to the horizontal and vertical projections, for exposure B, at h=30 ft (9.1m), I=1.0, and K_{zt} = 1.0. Adjust to other conditions using Equation 6-1.
2. The load patterns shown shall be applied to each corner of the building in turn as the reference corner. (See Figure 6-10)
3. For the design of the longitudinal MWFRS use $\theta = 0°$, and locate the zone E/F, G/H boundary at the mid-length of the building.
4. Load cases 1 and 2 must be checked for $25° < \theta \leq 45°$. Load case 2 at 25° is provided only for interpolation between 25° to 30°.
5. Plus and minus signs signify pressures acting toward and away from the projected surfaces, respectively.
6. For roof slopes other than those shown, linear interpolation is permitted.
7. The total horizontal load shall not be less than that determined by assuming $p_s = 0$ in zones B & D.
8. The zone pressures represent the following:
   Horizontal pressure zones – Sum of the windward and leeward net (sum of internal and external) pressures on vertical projection of:
   - A - End zone of wall
   - B - End zone of roof
   - C - Interior zone of wall
   - D - Interior zone of roof

   Vertical pressure zones – Net (sum of internal and external) pressures on horizontal projection of:
   - E - End zone of windward roof
   - F - End zone of leeward roof
   - G - Interior zone of windward roof
   - H - Interior zone of leeward roof
9. Where zone E or G falls on a roof overhang on the windward side of the building, use E_{OH} and G_{OH} for the pressure on the horizontal projection of the overhang. Overhangs on the leeward and side edges shall have the basic zone pressure applied.
10. Notation:
    a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).
    h: Mean roof height, in feet (meters), except that eave height shall be used for roof angles <10°.
    θ: Angle of plane of roof from horizontal, in degrees.

Figure 4-8: Design Wind Pressure Zones (Figure 6-2 in ASCE 7-05)

Table 4-21: Simplified Design Wind Pressure (from Figure 6-2 in ASCE 7-05)

Basic Wind Speed (mph)	Roof Angle (degrees)	Load Case	Zones									
			Horizontal Pressures				Vertical Pressures				Overhangs	
			A	B	C	D	E	F	G	H	EOH	GOH
85	0 to 5°	1	11.5	-5.9	7.6	-3.5	-13.8	-7.8	-9.6	-6.1	-19.3	-15.1
	10°	1	12.9	-5.4	8.6	-3.1	-13.8	-8.4	-9.6	-6.5	-19.3	-15.1
	15°	1	14.4	-4.8	9.6	-2.7	-13.8	-9.0	-9.6	-6.9	-19.3	-15.1
	20°	1	15.9	-4.2	10.6	-2.3	-13.8	-9.6	-9.6	-7.3	-19.3	-15.1
	25°	1	14.4	2.3	10.4	2.4	-6.4	-8.7	-4.6	-7.0	-11.9	-10.1
		2	------	------	------	------	-2.4	-4.7	-0.7	-3.0	------	------
	30 to 45	1	12.9	8.8	10.2	7.0	1.0	-7.8	0.3	-6.7	-4.5	-5.2
		2	12.9	8.8	10.2	7.0	5.0	-3.9	4.3	-2.8	-4.5	-5.2
90	0 to 5°	1	12.8	-6.7	8.5	-4.0	-15.4	-8.8	-10.7	-6.8	-21.6	-16.9
	10°	1	14.5	-6.0	9.6	-3.5	-15.4	-9.4	-10.7	-7.2	-21.6	-16.9
	15°	1	16.1	-5.4	10.7	-3.0	-15.4	-10.1	-10.7	-7.7	-21.6	-16.9
	20°	1	17.8	-4.7	11.9	-2.6	-15.4	-10.7	-10.7	-8.1	-21.6	-16.9
	25°	1	16.1	2.6	11.7	2.7	-7.2	-9.8	-5.2	-7.8	-13.3	-11.4
		2	------	------	------	------	-2.7	-5.3	-0.7	-3.4	------	------
	30 to 45	1	14.4	9.9	11.5	7.9	1.1	-8.8	0.4	-7.5	-5.1	-5.8
		2	14.4	9.9	11.5	7.9	5.6	-4.3	4.8	-3.1	-5.1	-5.8
100	0 to 5°	1	15.9	-8.2	10.5	-4.9	-19.1	-10.8	-13.3	-8.4	-26.7	-20.9
	10°	1	17.9	-7.4	11.9	-4.3	-19.1	-11.6	-13.3	-8.9	-26.7	-20.9
	15°	1	19.9	-6.6	13.3	-3.8	-19.1	-12.4	-13.3	-9.5	-26.7	-20.9
	20°	1	22.0	-5.8	14.6	-3.2	-19.1	-13.3	-13.3	-10.1	-26.7	-20.9
	25°	1	19.9	3.2	14.4	3.3	-8.8	-12.0	-6.4	-9.7	-16.5	-14.0
		2	------	------	------	------	-3.4	-6.6	-0.9	-4.2	------	------
	30 to 45	1	17.8	12.2	14.2	9.8	1.4	-10.8	0.5	-9.3	-6.3	-7.2
		2	17.8	12.2	14.2	9.8	6.9	-5.3	5.9	-3.8	-6.3	-7.2
105	0 to 5°	1	17.5	-9.0	11.6	-5.4	-21.1	-11.9	-14.7	-9.3	-29.4	-23.0
	10°	1	19.7	-8.2	13.1	-4.7	-21.1	-12.8	-14.7	-9.8	-29.4	-23.0
	15°	1	21.9	-7.3	14.7	-4.2	-21.1	-13.7	-14.7	-10.5	-29.4	-23.0
	20°	1	24.3	-6.4	16.1	-3.5	-21.1	-14.7	-14.7	-11.1	-29.4	-23.0
	25°	1	21.9	3.5	15.9	3.5	-9.7	-13.2	-7.1	-10.7	-18.2	-15.4
		2	------	------	------	------	-3.7	-7.3	-1.0	-4.6	------	------
	30 to 45	1	19.6	13.5	15.7	10.8	1.5	-11.9	0.6	-10.3	-6.9	-7.9
		2	19.6	13.5	15.7	10.8	7.6	-5.8	6.5	-4.2	-6.9	-7.9
110	0 to 5°	1	19.2	-10.0	12.7	-5.9	-23.1	-13.1	-16.0	-10.1	-32.3	-25.3
	10°	1	21.6	-9.0	14.4	-5.2	-23.1	-14.1	-16.0	-10.8	-32.3	-25.3
	15°	1	24.1	-8.0	16.0	-4.6	-23.1	-15.1	-16.0	-11.5	-32.3	-25.3
	20°	1	26.6	-7.0	17.7	-3.9	-23.1	-16.0	-16.0	-12.2	-32.3	-25.3
	25°	1	24.1	3.9	17.4	4.0	-10.7	-14.6	-7.7	-11.7	-19.9	-17.0
		2	------	------	------	------	-4.1	-7.9	-1.1	-5.1	------	------
	30 to 45	1	21.6	14.8	17.2	11.8	1.7	-13.1	0.6	-11.3	-7.6	-8.7
		2	21.6	14.8	17.2	11.8	8.3	-6.5	7.2	-4.6	-7.6	-8.7
120	0 to 5°	1	22.8	-11.9	15.1	-7.0	-27.4	-15.6	-19.1	-12.1	-38.4	-30.1
	10°	1	25.8	-10.7	17.1	-6.2	-27.4	-16.8	-19.1	-12.9	-38.4	-30.1
	15°	1	28.7	-9.5	19.1	-5.4	-27.4	-17.9	-19.1	-13.7	-38.4	-30.1
	20°	1	31.6	-8.3	21.1	-4.6	-27.4	-19.1	-19.1	-14.5	-38.4	-30.1
	25°	1	28.6	4.6	20.7	4.7	-12.7	-17.3	-9.2	-13.9	-23.7	-20.2
		2	------	------	------	------	-4.8	-9.4	-1.3	-6.0	------	------
	30 to 45	1	25.7	17.6	20.4	14.0	2.0	-15.6	0.7	-13.4	-9.0	-10.3
		2	25.7	17.6	20.4	14.0	9.9	-7.7	8.6	-5.5	-9.0	-10.3

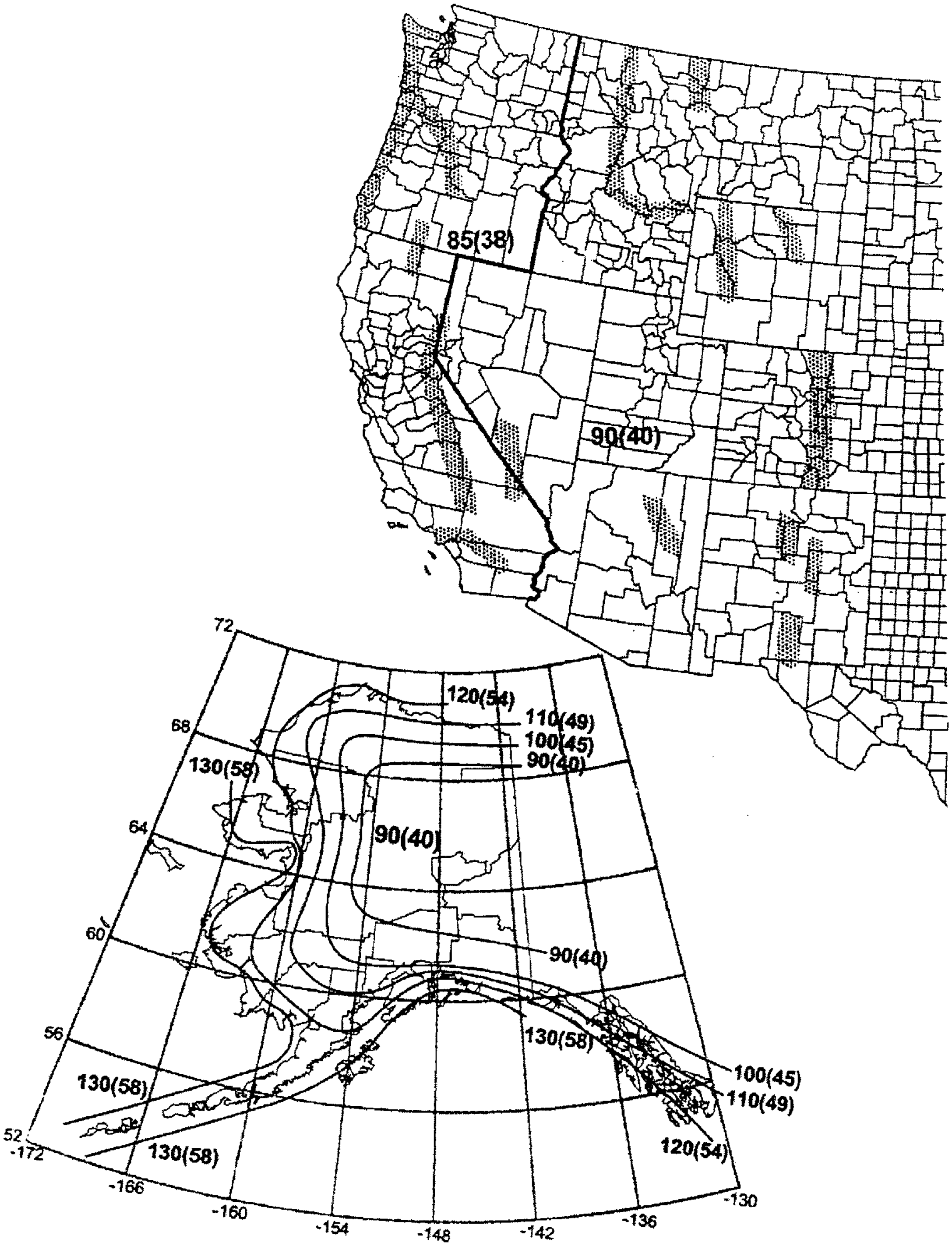
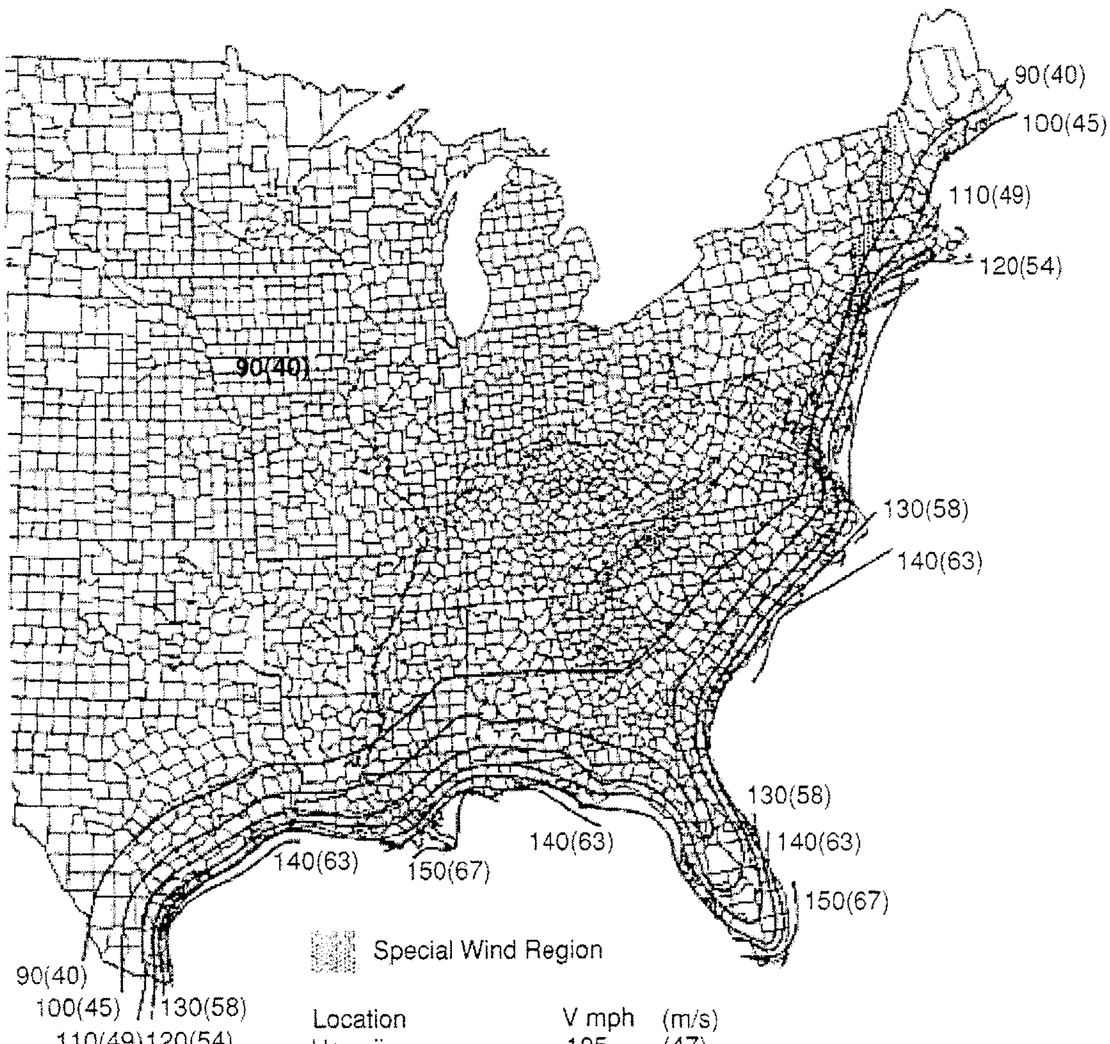


Figure 4-9: Basic Wind Speed (Figure 6-1 in ASCE 7-05)



Location	V mph	(m/s)
Hawaii	105	(47)
Puerto Rico	145	(65)
Guam	170	(76)
Virgin Islands	145	(65)
American Samoa	125	(56)

Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10 m) above ground for Exposure C category.
2. Linear interpolation between wind contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

Figure 4-10: Basic Wind Speed (Figure 6-1 in ASCE 7-05)

While the requirements in order to design a structure under Method 1 are specific, many low-rise structures will qualify. The more detailed analytical method will cover what the simplified method will not. Method 2 is significantly more detailed and complicated, and is beyond the scope of this thesis.

4.3 2006 International Building Code

By January 2008, the 2006 International Building Code will be the dominant code in 49 of the 50 United States (Hawaii has not yet adopted the 2006 IBC). Specifically, the structural section of the code will be identical throughout the 49 states. The 2008 California Building Code is not yet available for review; however, the seismic and wind sections of the 2008 CBC will be the same as what is described in the 2006 IBC.

4.3.1 **Seismic**

The first part of the scope of the section on earthquake loads (Section 1613) in the 2006 IBC reads as follows:

> Every Structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7...

Similarly to the 2003 IBC, the 2006 IBC accepts the seismic design detailed in the ASCE 7, but instead of including some of the equations, tables and figures in the pages of the code, the 2006 IBC refers the reader directly to the ASCE 7.

While the wording and some arrangements are different, the two codes are identical since they both refer to the same building standard. All of the procedures, figures and tables are largely the same. The biggest difference between the 2003 and 2006 version exists among the maximum ground motion contour maps.

These maps are more detailed in the 2006 version, especially on the west coast. Also, there are expanded sections for two sections of the country in the 2003 IBC, but four in the 2006 version. The most significant difference between the two sets of contour maps is that the values on the 2006 contours for most of the country are larger than the corresponding values on the 2003 maps, and smaller for California. Also, the fault lines in California are fewer in the 2006 version. The large national 2006 IBC contour maps can be seen in Figure 4-11 (S_S for the western US), Figure 4-12 (S_S for the eastern US), Figure 4-13 (S_1 for the western US) and Figure 4-14 (S_1 for the eastern US).

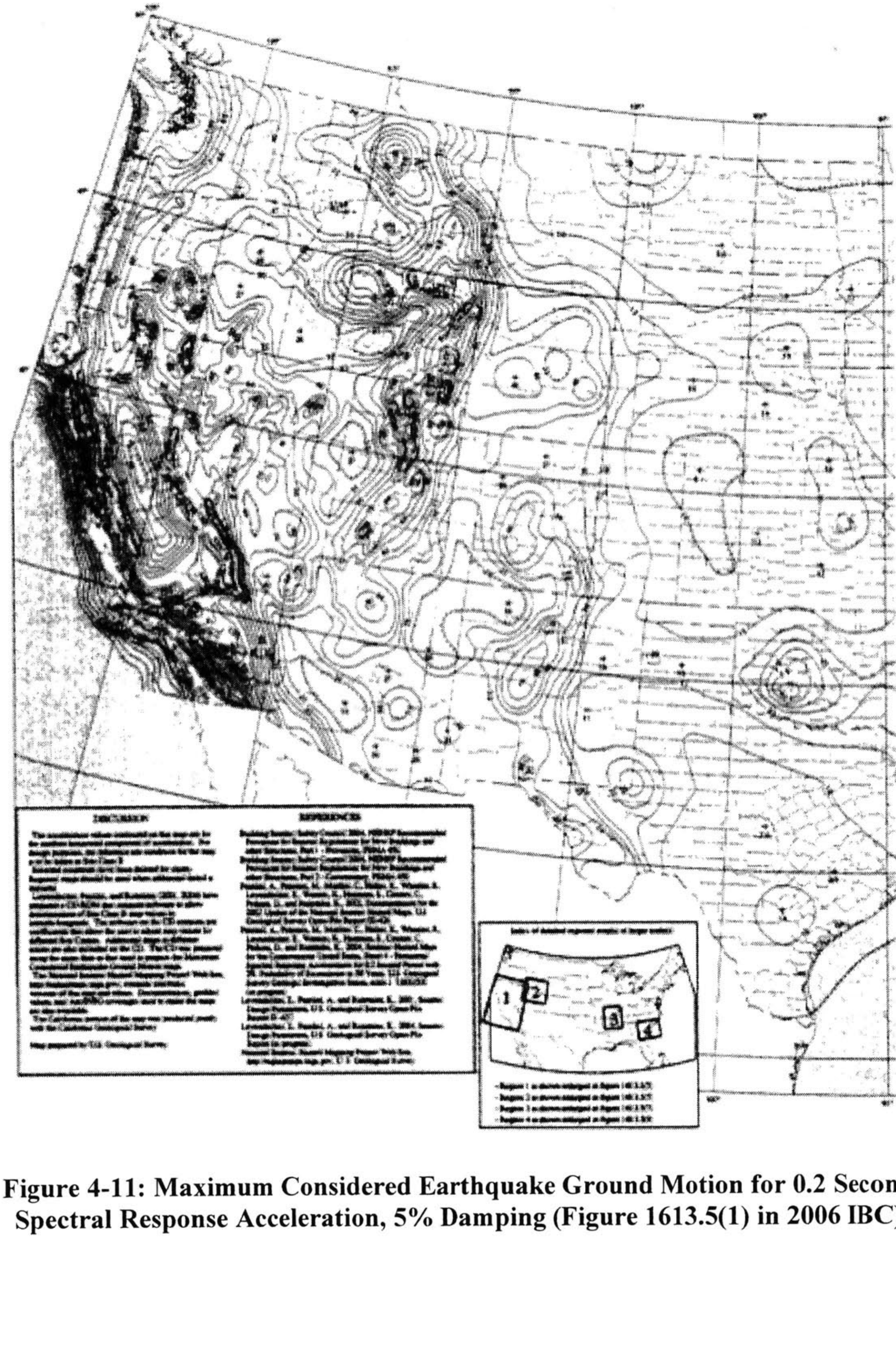


Figure 4-11: Maximum Considered Earthquake Ground Motion for 0.2 Second Spectral Response Acceleration, 5% Damping (Figure 1613.5(1) in 2006 IBC)

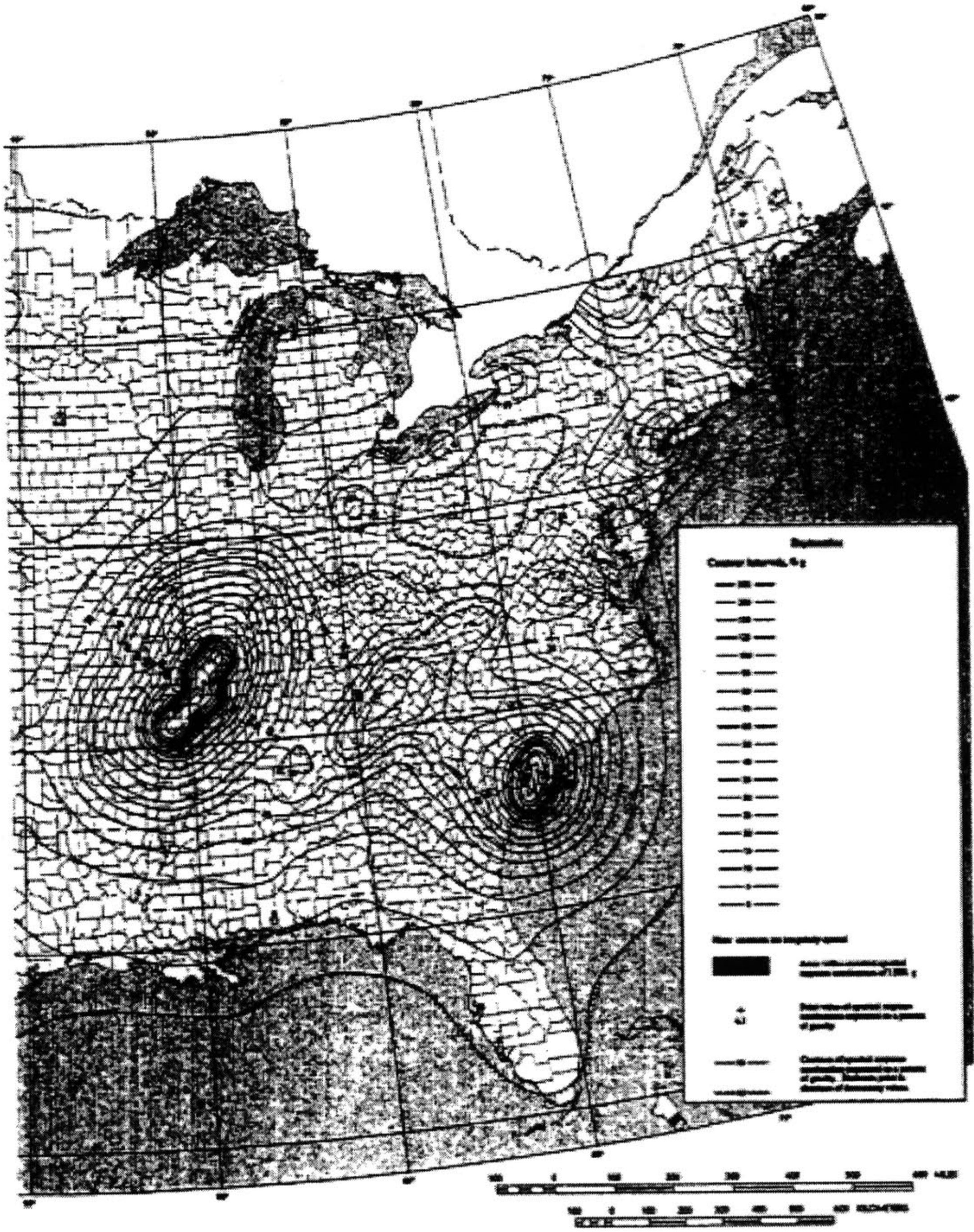


Figure 4-12: Maximum Considered Earthquake Ground Motion for 0.2 Second Spectral Response Acceleration, 5% Damping (Figure 1613.5(1) in 2006 IBC)

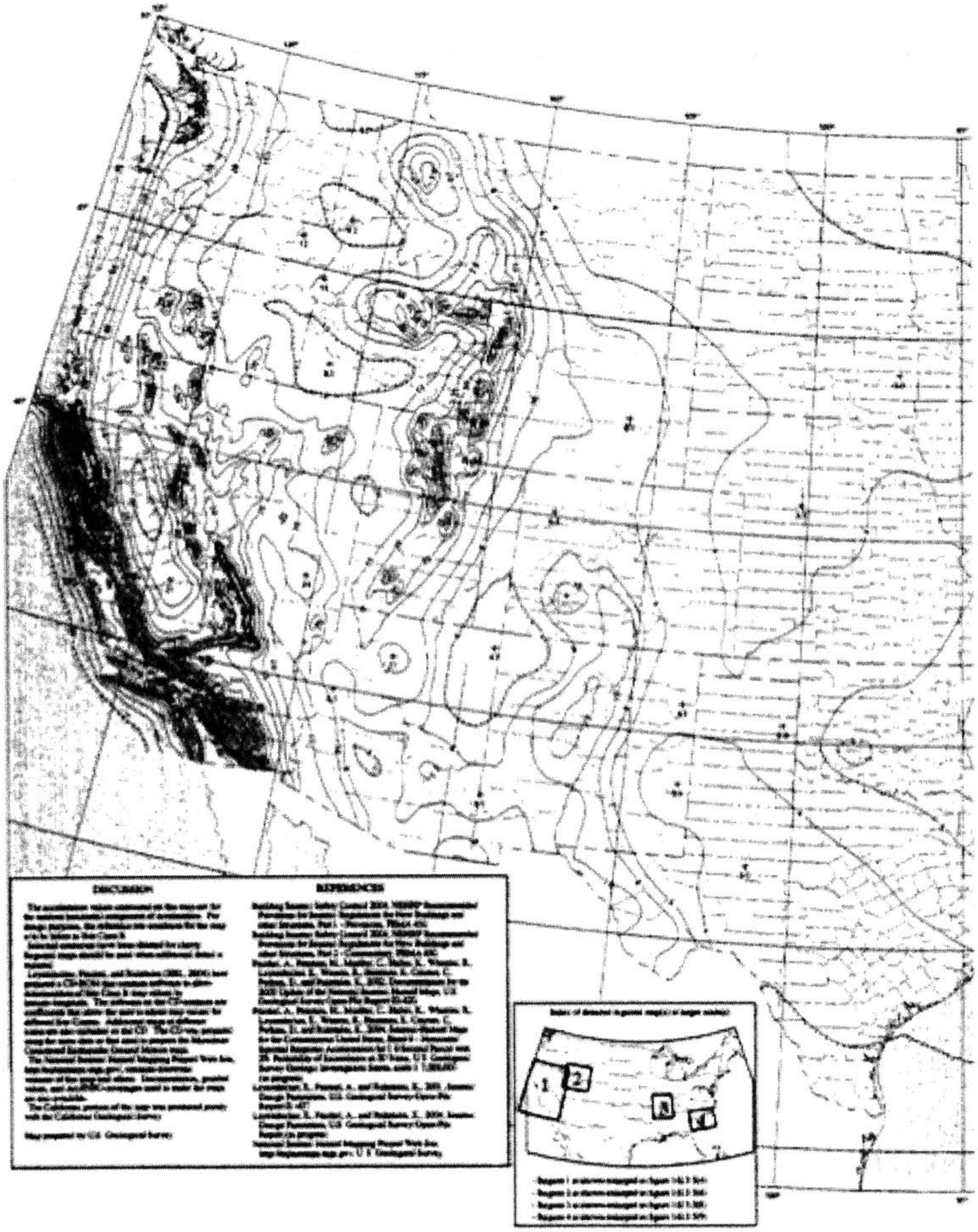


Figure 4-13: Maximum Considered Earthquake Ground Motion for 1.0 Second Spectral Response Acceleration, 5% Damping (Figure 1613.5(2) in 2006 IBC)

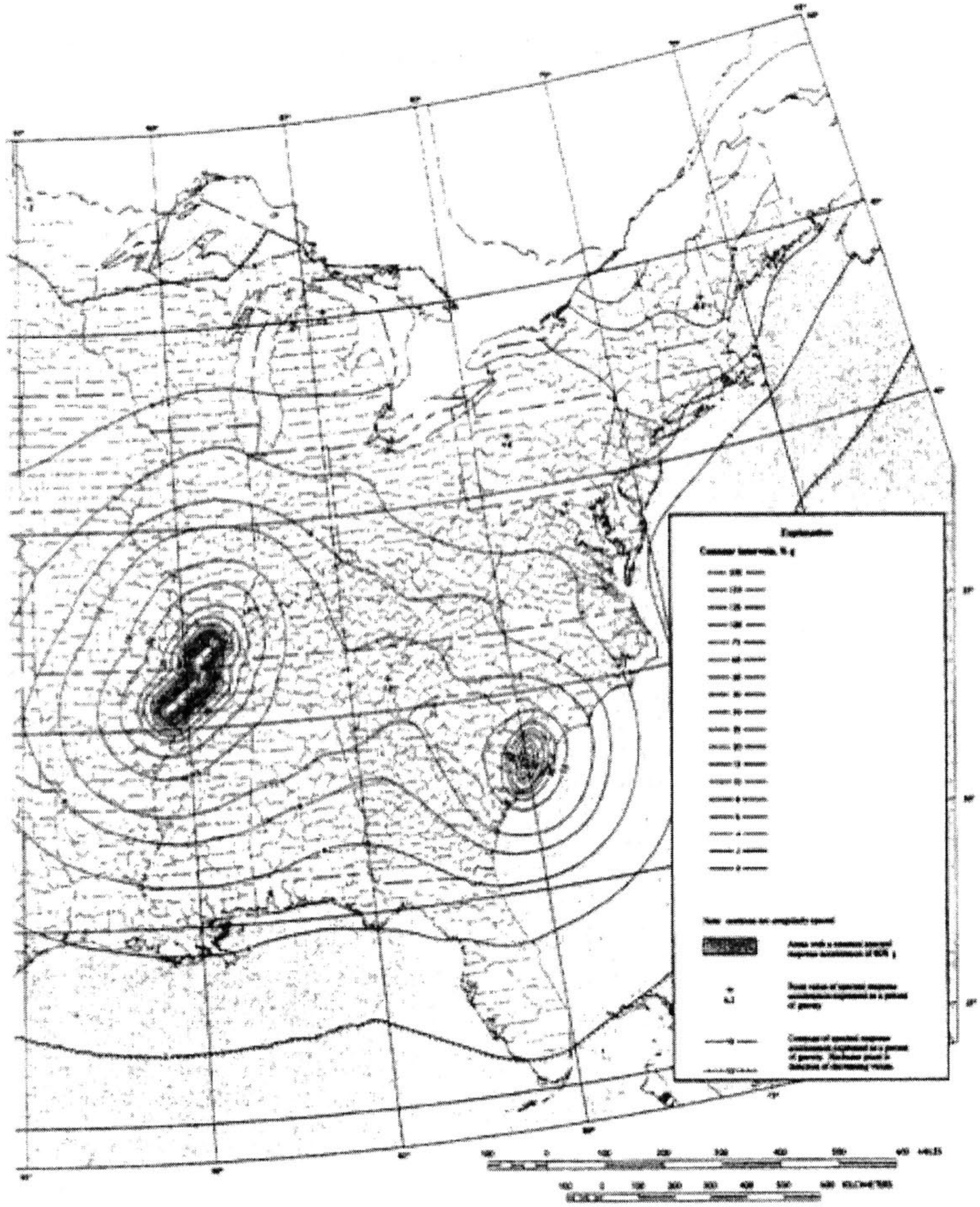


Figure 4-14: Maximum Considered Earthquake Ground Motion for 1.0 Second Spectral Response Acceleration, 5% Damping (Figure 1613.5(2) in 2006 IBC)

While the procedure in the 2006 IBC is similar to the 2003 IBC, the ground motion differences will lead to different bas shear values. In addition, the design response spectrum is constructed slightly differently than in the 2006 IBC. This is shown in Figure

4-15. As can be seen in the figure, the 2006 method for constructing the response spectrum only differs from the 2003 method for high periods, specifically for periods greater than the long-period transition period, T_L. Like the 2003 model, the interval between T_0 and T_S is where constant acceleration occurs. The constant velocity interval is now between T_S and T_L. The portion of the plot with periods greater than T_L represents where the response can be expected to have constant displacement. The spectral response acceleration declines more rapidly as the fundamental period of a structure increases in the 2006 IBC and compared to the 2003 IBC. Structures with such high periods will not be as affected by ground motion during an earthquake, so a less conservative design is necessary. This section was not included in the 2003 version because it was assumed that very few structures would have long enough periods to fall under this portion of the spectrum, however, it has been added to the 2006 IBC.

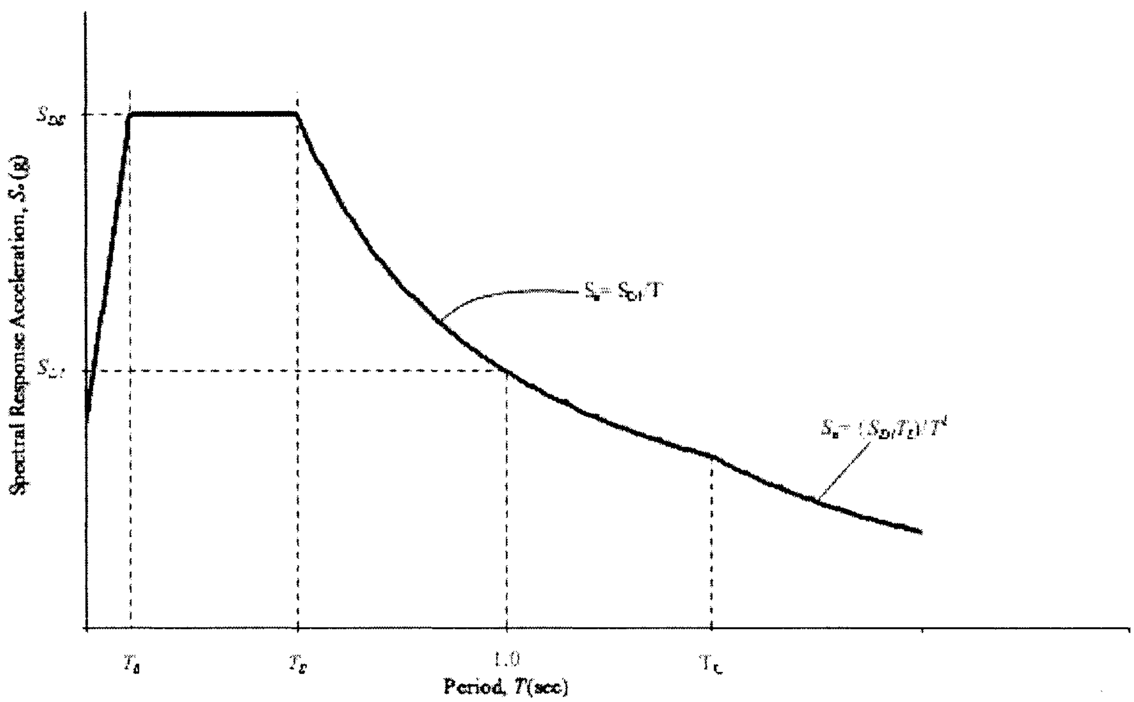


Figure 4-15: Design Response Spectrum (Figure 11.4.1 in ASCE 7-05)

The long-period transition period of a structure, T_L, is determined from standardized contour maps. The resulting value is based on the region in which a building site is located. An example of one of these maps can be found in Figure 4-16 and Figure 4-17, which show the western and eastern portion of the United States, respectively.

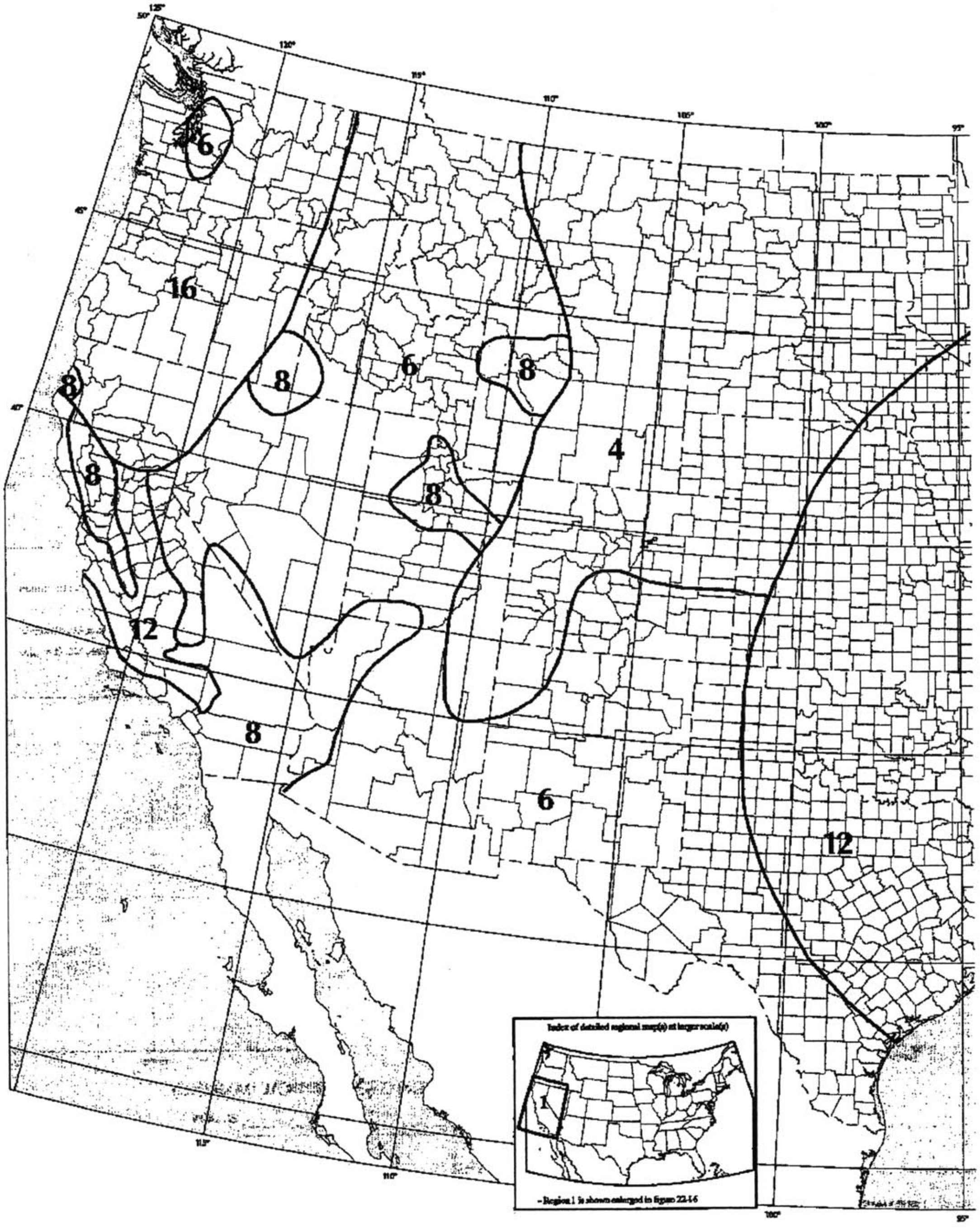


Figure 4-16: Long-Period Transition Period (from Figure 22-15 in ASCE 7-05)

Figure 4-17: Long-Period Transition Period (from Figure 22-16 in ASCE 7-05)

With the inclusion of this new section of the response spectrum, it became necessary to include an equation for the maximum seismic coefficient, Cs, for structures that have periods greater than T_L. This equation is shown in Equation 4-20.

$$C_s = \frac{S_{D1} T_L}{T^2 (\frac{R}{I})} \qquad \text{for } T > T_L$$

Equation 4-20: Cs Maximum for Long T (Equation 12.8-4 in ASCE 7-05)

where: S_{D1} is the design spectral response acceleration parameter at a period of 1.0 second
R is the response modification factor
I is the occupancy importance factor
T is the fundamental period of the structure, in seconds
T_L is the long-period transition period, in seconds

With the exception of these few changes, the earthquake design loads are determined the same way as in the 2003 IBC.

4.3.2 Wind

Like the seismic section of the 2006 IBC, there are no major changes between the 2003 and 2006 versions.

4.4 Highlights of Differences and Why the Changes are Occurring

As discussed in Section 4.2, the differences between the 2003 IBC and 2006 IBC are few. The two important changes between the two versions are the updated ground motion contour maps (Figure 4-3 through Figure 4-6 for 2003 and Figure 4-11, through Figure 4-14 for 2006) and the alteration to the spectral acceleration response plot (Figure 4-7 for 2003 and Figure 4-15 for 2006). As more information becomes available, the contour maps will be updated, so it is logical that these maps would show some changes in each new version of the IBC.

The spectral acceleration plots are derived from empirical earthquake data that relate acceleration to period. As more empirical data becomes available, or as a curve is more accurately fitted to the data, this plot will change. As mentioned in Section 4.2, the difference between the 2003 and 2006 versions of this plot occurs where the fundamental period is greater than the long transition period, T_L. Earthquake motion will be transmitted less to structures with longer periods, and this is reflected in the new 2006 version of the plot, and in the equations that resulted from including this factor.

As shown in Section 4.1, the methods employed by the 2001 CBC significantly differ from those used in the 2006 IBC. The main reason for abandoning the methods used in the CBC and adopting those in the IBC is to unify the country under one building code. California is the last continental state to conform to the IBC. Once the entire country uses the IBC, then the code will have to opportunity to stretch beyond the US borders into other countries. In addition, the shift in the 2006 IBC from seismic categories to the use of the ground motion contour maps will help the code to expand out of the United States. With the methods in the IBC, the only additional information required in order to use the code outside the US would be expected ground motion contour maps, long transition period maps and maximum wind speed maps. Standardizing the procedure makes it possible for the IBC to have universal applications with a minimal amount of necessary additional information. In time, the IBC may live up to its name and become the International building code.

5 Comparison of Design Results From Each Code

The focus of this thesis is to evaluate the possible impacts of the move from an older version of building code to the 2006 IBC. An effective way to evaluate differences in the codes is to compare design results from old and new versions of the code. In order to illustrate the differences, the following sections will detail design examples.

In order to analyze the changes to the seismic code, a design was conducted in two parts of the country, and the resulting design base shear values were compared. The reason for considering two locations is to look at both a high and a low seismic area, since the code changes may have different effects in these different areas. For the earthquake-prone area, a site in Southern California is considered; this is done in section 5.1. Designs from the 2001 CBC, 2003 IBC and IBC were carried out in sections 5.1.2, 5.1.3 and 5.1.4, respectively, and the comparison is in section 5.1.5. The purpose of comparing both the 2003 IBC and the 2001 CBC to the 2006 IBC is to show the effects the new IBC. For the low seismic area, the same design example is carried out in the Midwest; this is done in section 5.2. In this area the code will change from the 2003 IBC to the 2006 IBC, so those are the methods carried out in sections 5.2.2 and 5.2.3, respectively.

To compare the wind code changes, in section 5.3, it was only necessary to analyze one site since only California will endure a significant wind code change. The same structure used in the seismic examples is used for this wind example, and the resulting design wind pressures were compared. The location chosen for this example is in Northern California where seismic activity is not as prevalent as in the southern part of the state. For this design, the 2001 CBC, shown in section 5.3.2, is compared to the 2006 IBC, done in section 5.3.3, to illustrate the effects the new IBC will have on the wind loading design of structures in California.

5.1 Seismic: California Site

This section is concerned with a structure in California (intersection of 116° longitude and 34° latitude). The approximate site is also shown on a map of California in Figure 5-1.

5.1.1 Design Assumptions

The parameters listed here are assumptions that are common to all of the designs that are carried in this chapter. The structure will be a low-rise (three stories, mean roof height of 45 feet) with a regular footprint (100ft x 100ft) and standard occupancy. The structural system is assumed to be an ordinary steel moment-resisting frame. The site is assumed to be a "stiff soil profile" which is what is to be assumed if no information about the site is known. The total effective seismic weight of the structure was calculated assuming 100 psf of dead load and a total floor area of 30,000 ft^2 (3 100ft x 100ft floors), which equals 3,000 kips. Further specific assumptions, beyond the common ones listed here, for each of the two locations will be detailed below.

5.1.2 2001 CBC Design

Since the structure is theoretically located in the earthquake-prone southern California, the seismic zone factor, Z, (Figure 4-1 and Table 4-1) is 0.40. Similarly to the IBC designs, the importance factor from Table 4-3 is 1.0. R, the structural system factor (Table 4-4), is 4.5 and C_t, the period factor, is 0.035 since the structure is a steel moment-resisting frame. Using Equation 4-5 the period is calculated to be 0.61 seconds. Since the site is in seismic zone 4, the seismic source type is important. This part of California has numerous faults that can produce high magnitude events, so the source type will be assumed to be B, and considered close to the building site. Figure 5-1 shows a map of California where the grey lines correspond to fault lines, and the "X" is approximately the location of the site.

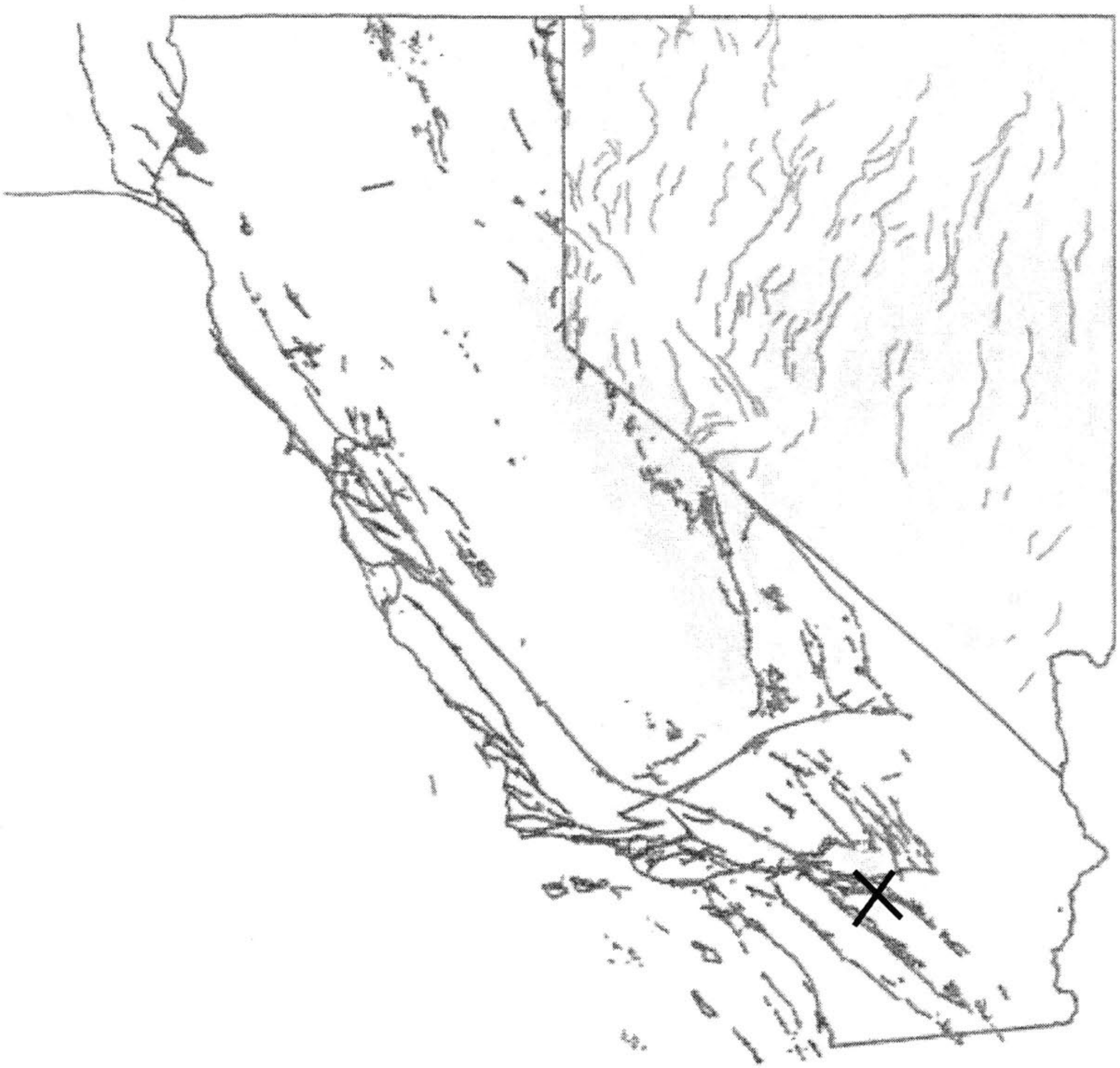


Figure 5-1: California Faults (USGS)

Using these assumptions, the near-source factors, N_v (Table 4-6) and N_a (Table 4-8), are 1.6 and 1.3, respectively. Using the site classification, S_D, the near-source factors and the seismic zone factor, the seismic coefficients, Cv (Table 4-5) and Ca (Table 4-6), are 1.024 and 1.6, respectively. Next the base shear, as well as the maximum and minimum base shears were calculated using Equation 4-1, Equation 4-2 and Equation 4-4. As with the IBC designs, the maximum controlled, and the corresponding base shear is 953.3 kips. The results are shown in the Table 5-1.

Table 5-1: 2001 CBC California Seismic Design Results

h	45	ft			
R	4.5				
I	1		W	3000	kip
Ct	0.035	ft	T	0.608	s
Site	SD		V	1122.6	lbs
Z	0.52		Vmax	953.3	kip
Na	1.3		Vmin	443.7	kip
Nv	1.6				
Ca	0.572				
Cv	1.024				

5.1.3 2003 IBC Design

Using the assumptions described above the design parameters relevant to the procedure detailed in the 2003 IBC were determined. Using Table 4-14 and using a standard occupancy, the Importance Factor, I, was determined to be 1.0. Since the structure will be an ordinary steel moment-resisting frame, period parameters C_t and x were determined to be 0.028 and 0.8, respectively (Table 4-16), and the response modification factor, R, is 3.5. Using the values for C_t and x and Equation 4-11, the fundamental period of the structure is approximately 0.59 seconds.

Next, the spectral accelerations, S_s and S_1, were determined from the contour maps that zoom in on southern California. These can be seen in Figure 5-2 (S_S Northern California), Figure 5-3 (S_S Southern California), Figure 5-4 (S_I Northern California) and Figure 5-5 (S_1 Southern California).

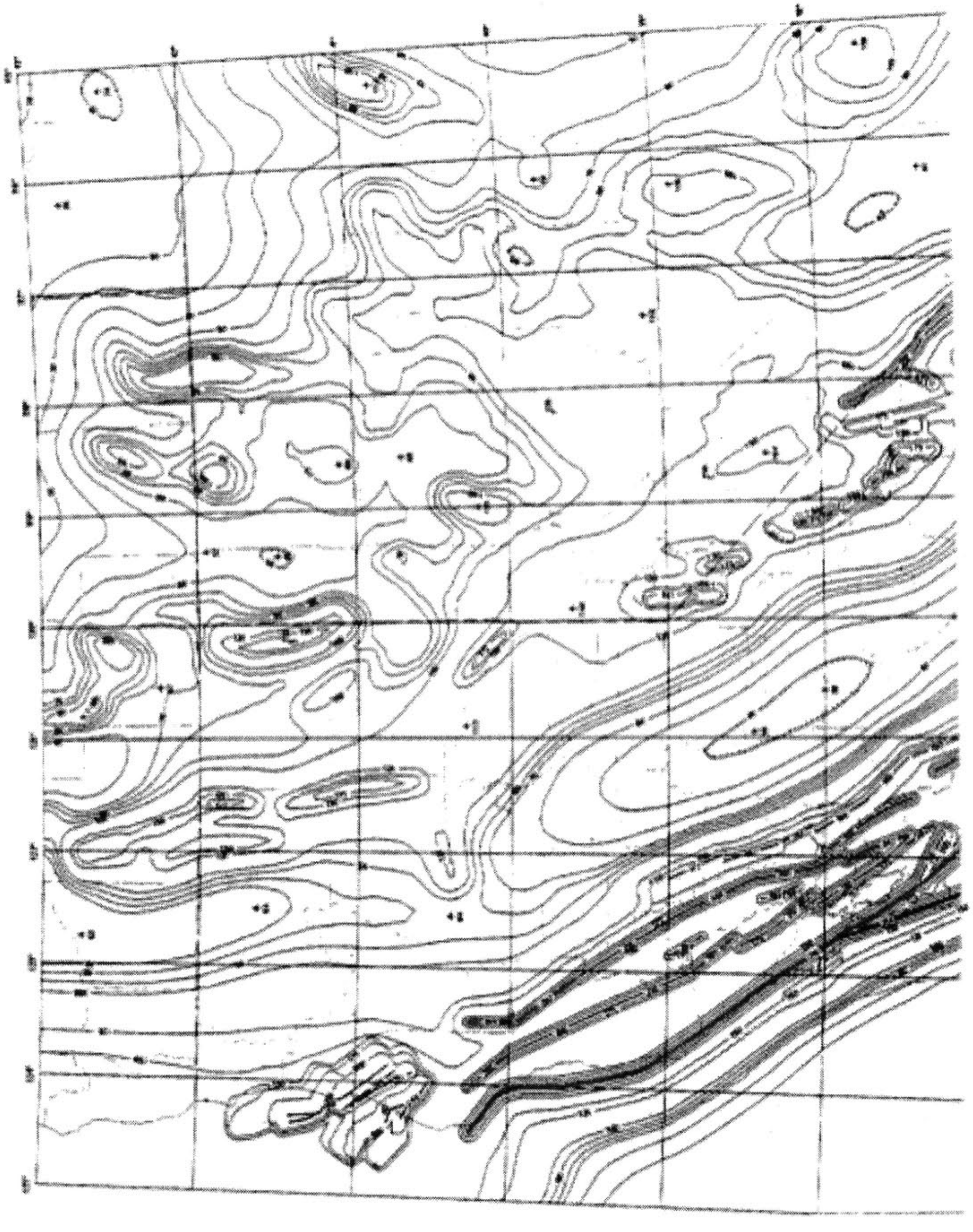


Figure 5-2: Maximum Considered Ground Motion for California of 0.2 Second Spectral Response Acceleration (5% Damping) (Figure 1615(3) in 2003 IBC)

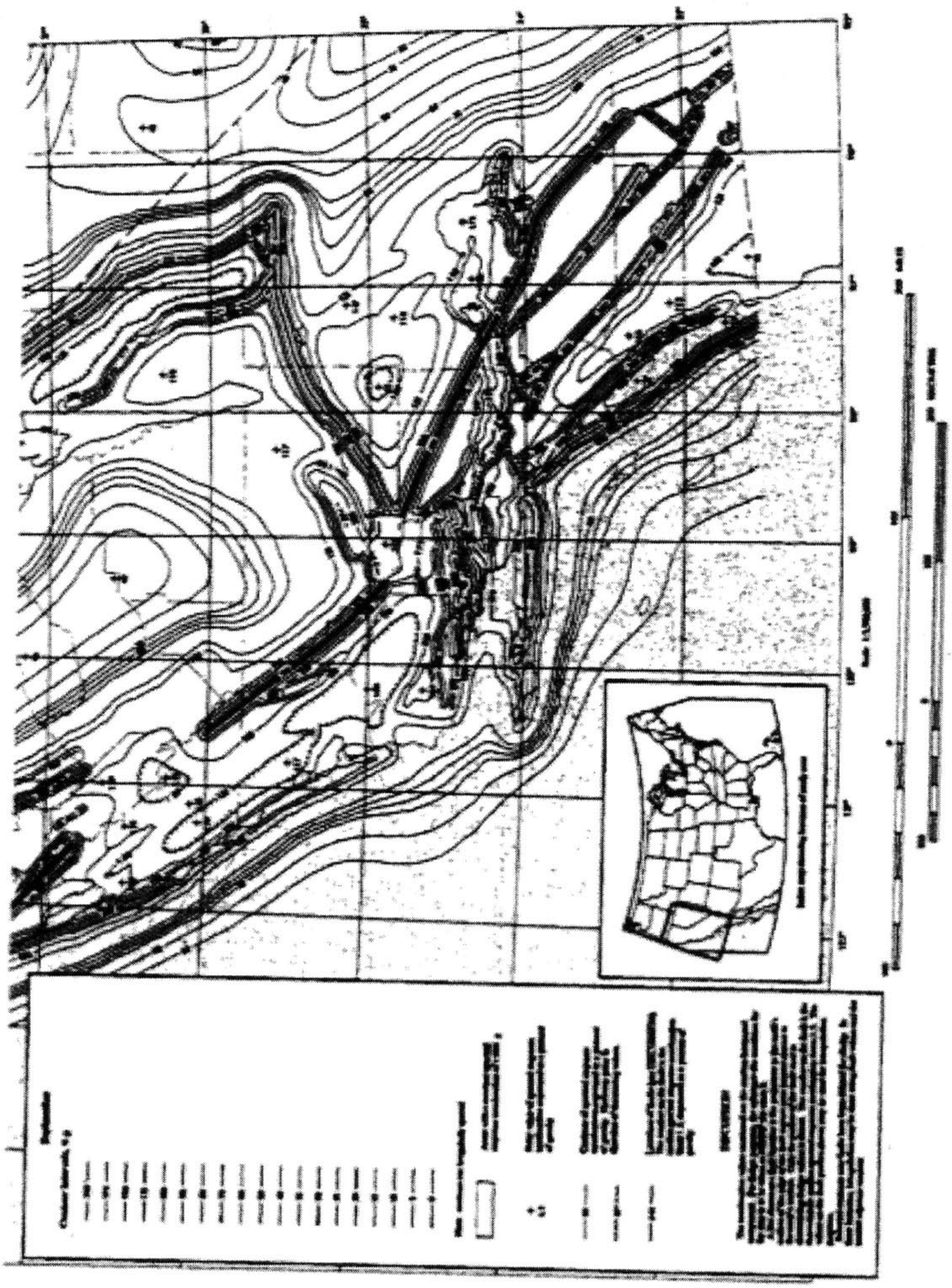


Figure 5-3: Maximum Considered Ground Motion for California of 0.2 Second Spectral Response Acceleration (5% Damping) (Figure 1615(3) in 2003 IBC)

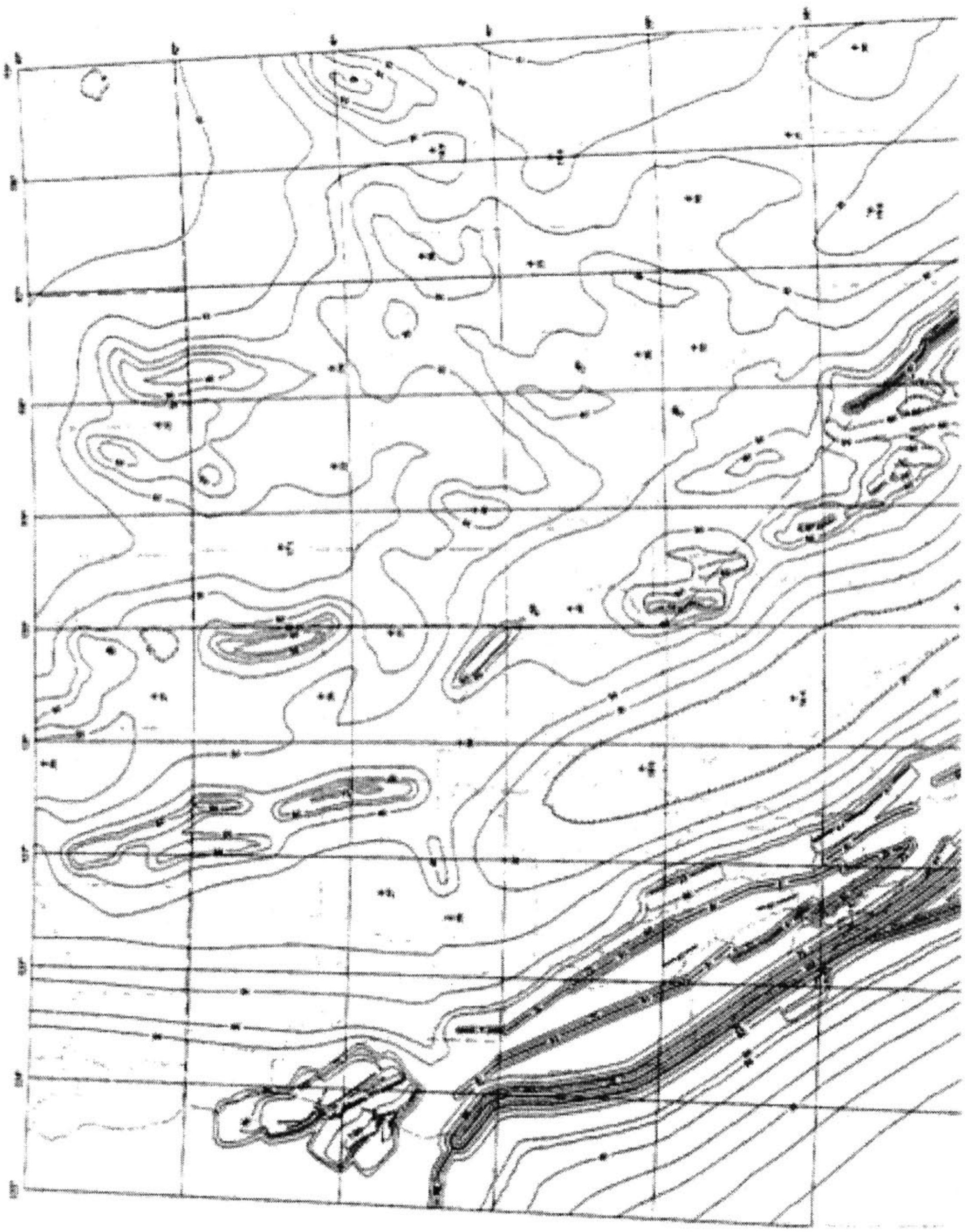


Figure 5-4: Maximum Considered Ground Motion for California of 1.0 Second Spectral Response Acceleration (5% Damping) (Figure 1615(4) in 2003 IBC)

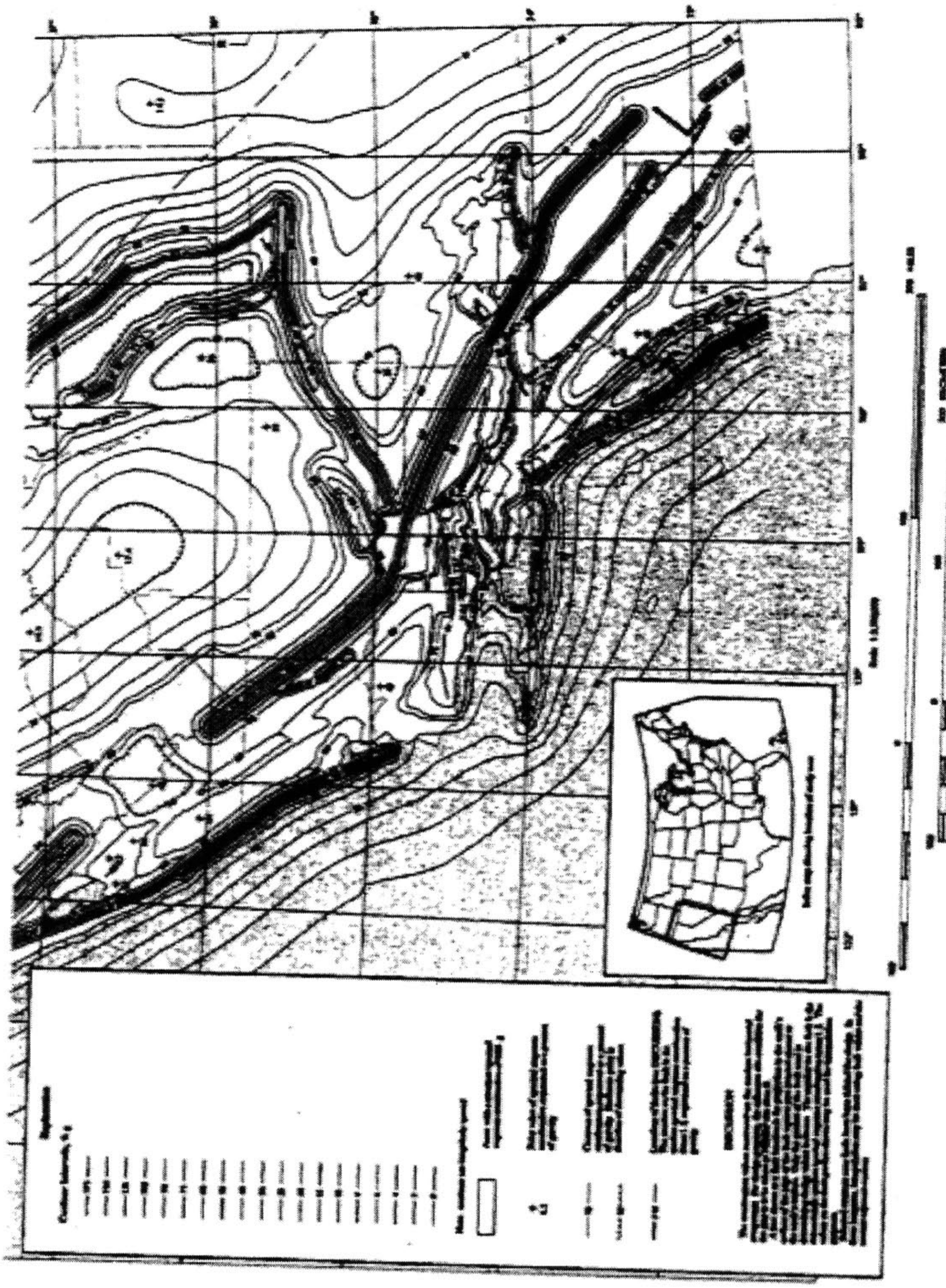


Figure 5-5: Maximum Considered Ground Motion for California of 1.0 Second Spectral Response Acceleration (5% Damping) (Figure 1615(4) in 2003 IBC)

As can be seen in the figures, the short and 1-second spectral accelerations for this location are 125%g and 54%g, respectively. With these values, the assumed site class, D, and Table 4-17 and Table 4-18, the site coefficients, F_a and F_v, can be determined (using interpolation) as 1.0 and 0.028. Inserting the site coefficients and the spectral accelerations into Equation 4-14 and Equation 4-15 gives maximum considered spectral accelerations of 1.35 and 0.96 for the short and 1-second periods, respectively. The next step is to determine S_{DS} and S_{D1}, the damped spectral response accelerations, using Equation 4-12 and Equation 4-13. Doing this, S_{DS} and S_{D1} come out to 0.833 and 0.708, respectively.

Using the parameters determined above and Equation 4-8, the seismic response coefficient, C_S, can be calculated as 0.405. In addition, the maximum C_S can be determined using Equation 4-9 to be 0.344, and the minimum C_S is 0.01. Since C_S exceeds the maximum C_S, the base shear will be determined using C_S(max). With Equation 4-7 and the assumed structure weight, the base shear is approximately 1031.9 kips. All of the variables and results can be seen in Table 5-2.

Table 5-2: 2003 IBC California Seismic Design Results

h	45	ft	W	3000	kip
R	3.5		S_{MS}	1.250	
I	1		S_{M1}	1.063	
C_t	0.028	ft	S_{DS}	0.833	
x	0.8		S_{D1}	0.708	
Site	D		T	0.588	
S_S	125	%g	C_S	0.405	
S_1	54	%g	C_s min	0.077	
F_a	1		C_s max	0.344	
F_v	1.968		V	1031.9	kip

5.1.4 2006 IBC Design

Since the seismic design procedure and equations did not change from the 2003 to the 2006 IBC the above procedure is the same for the 2006 design of the structure. However, the mapped spectral accelerations did change, so while the equations did not change, the results will. The contours from the 2006 IBC are shown in Figure 5-6 (S_S for Northern California), Figure 5-7 (S_S for Southern California), Figure 5-8 (S_1 Northern California) and Figure 5-9 (S_1 for Southern California).

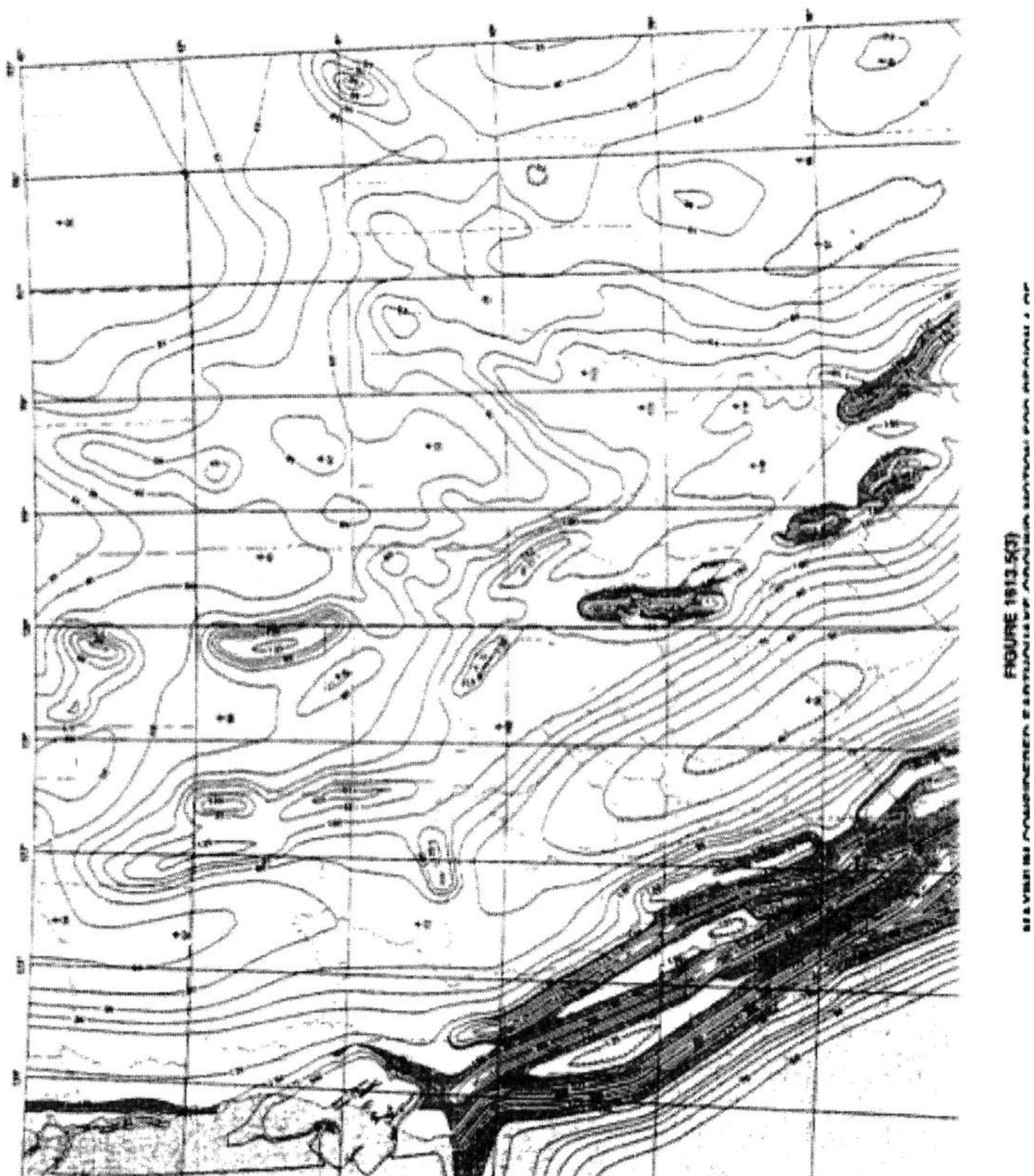


Figure 5-6: Maximum Considered Ground Motion for California of 0.2 Second Spectral Response Acceleration (5% Damping) (Figure 1613.5(3) in 2006 IBC)

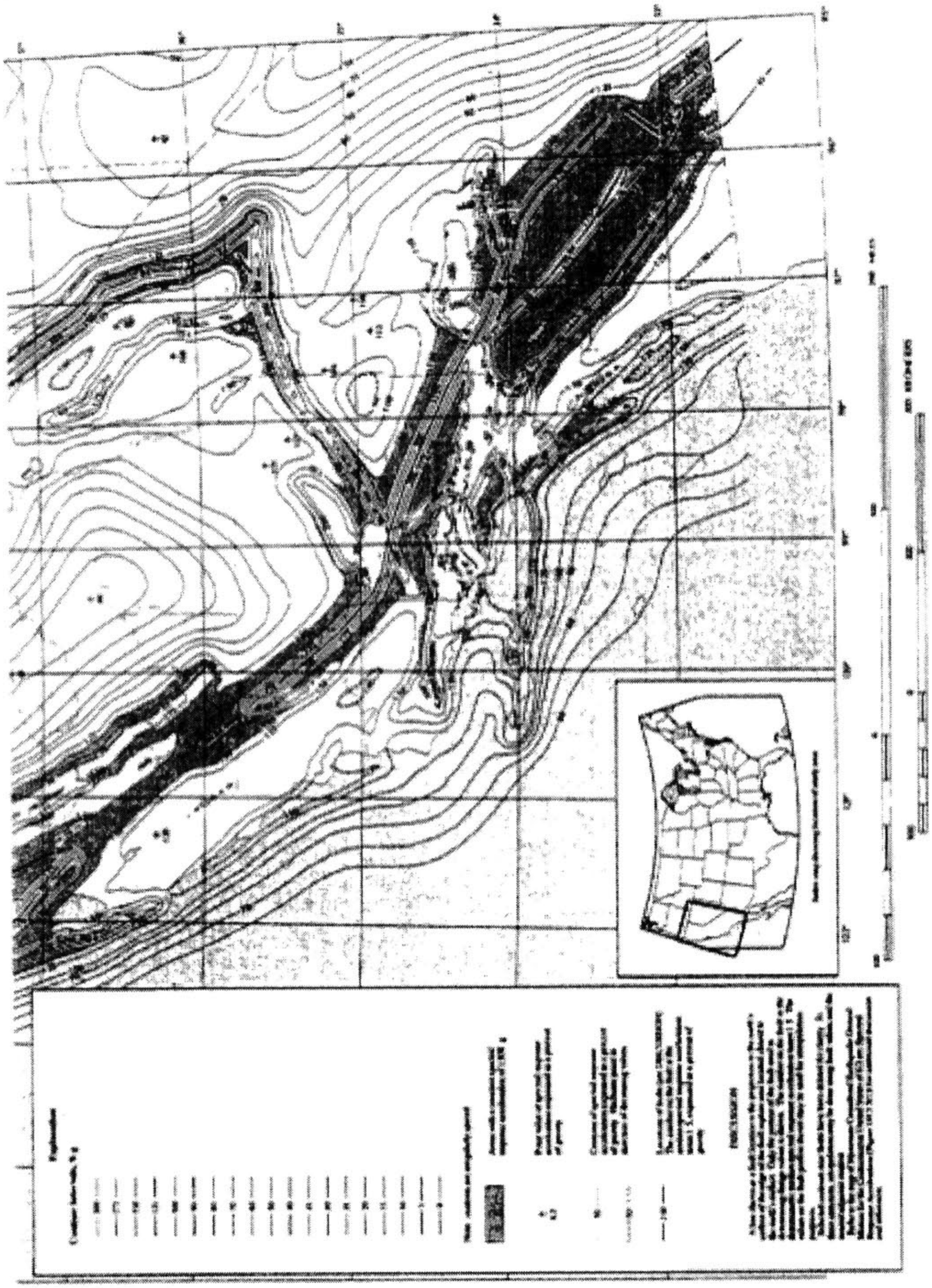


Figure 5-7: Maximum Considered Ground Motion for California of 0.2 Second Spectral Response Acceleration (5% Damping) (Figure 1613.5(3) in 2006 IBC)

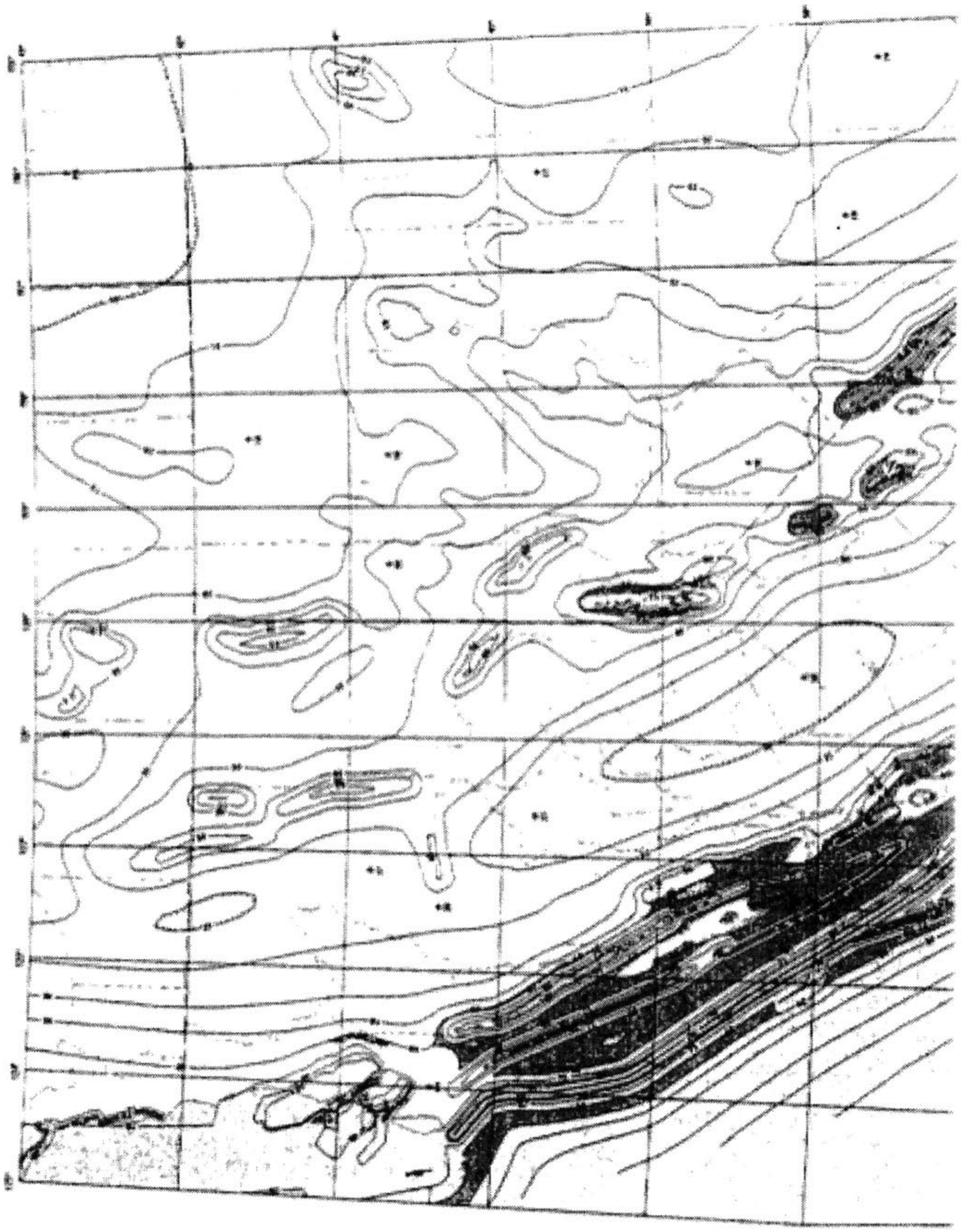


Figure 5-8: Maximum Considered Ground Motion for California of 1.0 Second Spectral Response Acceleration (5% Damping) (Figure 1613.5(4) in 2006 IBC)

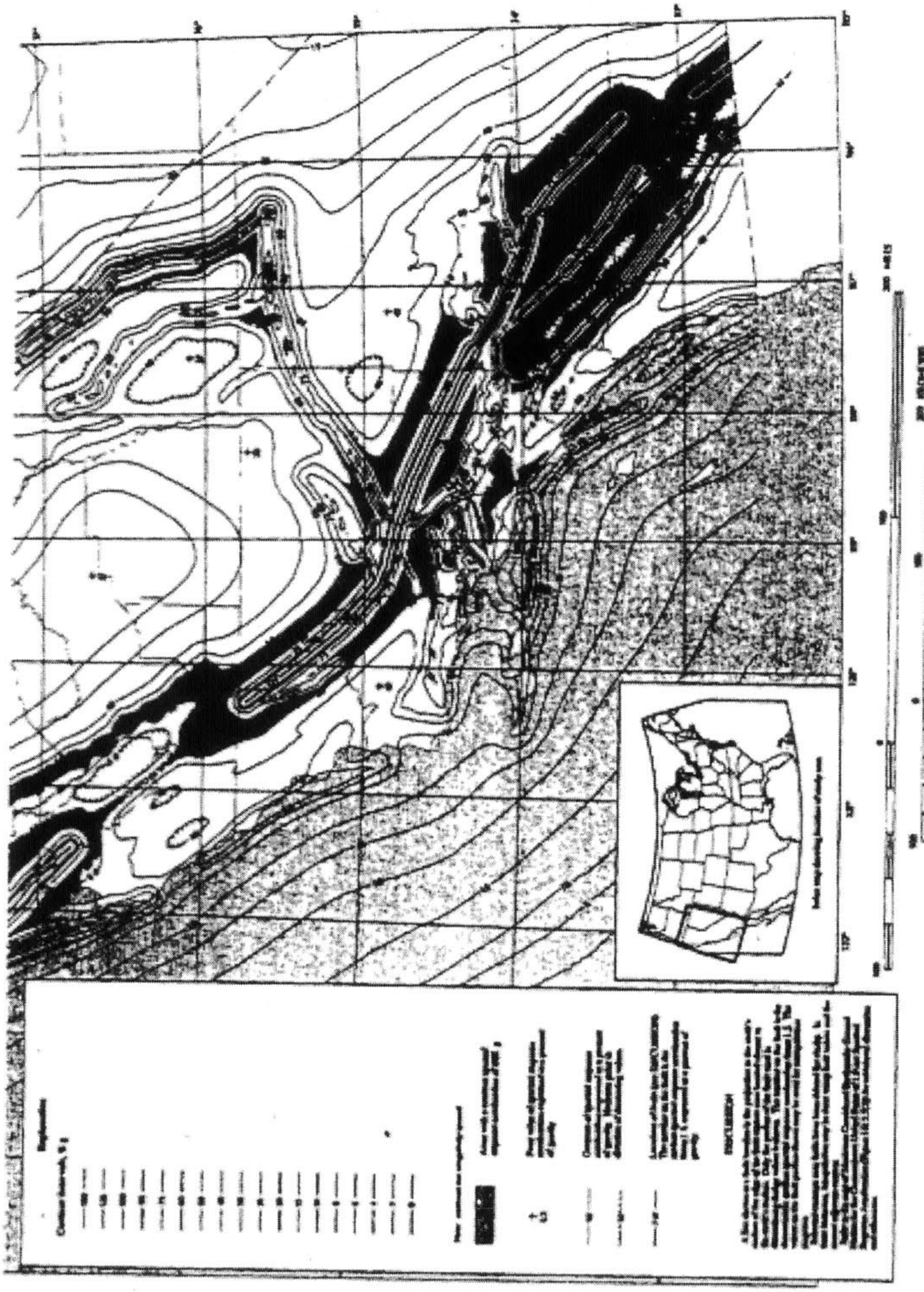


Figure 5-9: Maximum Considered Ground Motion for California of 1.0 Second Spectral Response Acceleration (5% Damping) (Figure 1613.5(4) in 2006 IBC)

While the maps are not drastically different, the short and 1-second mapped spectral accelerations were 135%g and 47%g, respectively. Like the 2003 IBC procedure, the C_S(max) controlled. Using the procedure and equations previously explained, the seismic response coefficient is 0.312 and the base shear is 934.7 kips. The results are shown in Table 5-2

Table 5-3: 2006 IBC California Seismic Design Results

h	45	ft	W	3000	kip
R	3.5		S_{MS}	1.350	
I	1		S_{M1}	0.963	
C_t	0.028	ft	S_{DS}	0.900	
x	0.8		S_{D1}	0.642	
Site	D		T	0.588	s
S_s	135	%g	C_S	0.437	
S_1	47	%g	C_s min	0.010	
F_a	1		C_s max	0.312	
F_v	2.048		V	934.7	kip

5.1.5 Comparisons and Discussion

Table 5-4 shows design parameters and results for all three designs. The bolded values are some important parameters that differed between the codes even though all the designs were based on the same basic assumptions.

Table 5-4: California Seismic Design Comparisons

	2001 CBC	2003 IBC	2006 IBC
h (ft)	45	45	45
R	**4.5**	**3.5**	**3.5**
I	1	1	1
C_t	**0.035**	**0.028**	**0.028**
x	-	0.8	0.8
Site Class	S_D	D	D
S_S (%g)	-	**125**	**135**
S_1 (%g)	-	**54**	**47**
F_a	-	1	1
F_v	-	1.968	2.048
Z	0.52	-	-
N_a	1.3	-	-
N_v	1.6	-	-
C_a	0.572	-	-
C_v	1.024	-	-
T (sec)	**0.608**	**0.588**	**0.588**
W (kip)	3000	3000	3000
S_{MS} (%g)	-	1.250	1.350
S_{M1} (%g)	-	1.063	0.963
S_{DS} (%g)	-	0.833	0.900
S_{D1} (%g)	-	0.708	0.642
C_s, value used	-	**0.344**	**0.312**
V, value used (kip)	**953.3**	**1031.9**	**934.7**
% diff (from 2001 CBC)	-	**+8.25%**	**-1.96%**
% diff (from 2003 IBC)	-	-	**-10.4%**

The percent differences are based on the variation of the base shear, V, calculated from each design. The CBC and 2003 IBC values show percent differences as compared with the 2006 design. The basic equation is shown in Equation 5-1.

$$\%diff = \frac{New - Old}{Old} * 100\%$$

Equation 5-1: Percent Difference

where: "Old" refers to the base shear, V, from either the CBC or 2003 IBC
"New" refers to the base shear, V, from the 2006 IBC

Although the 2003 and 2006 IBC procedures were the same, the results showed a decrease of 10.4% in design base shear from the 2003 IBC to the 2006 IBC, and a 8.25% decrease from the 2001 CBC to the 2003 IBC. These changes in the design base shear will change the design of the lateral force resisting system. This comparison was carried out to show what the design values would have been if California chose to adopt the 2003 version of the IBC. As can bee seen from Table 5-4, the 2003 IBC design value is more conservative than the other two, so the effects on California design values would have become more conservative if California would have previously chosen to adopt the 2003 IBC. The 2006 IBC values are much more consistent with the existing CBC values.

The difference between the 2001 CBC and the 2006 IBC design base shear was a decrease of about 2 percent. While this is not as large as the difference between the old and new IBC, it is still significant. In addition, other areas of California may have more significant changes. However, the design shear from the IBC is smaller than that of the CBC. This will mean a relaxation of lateral designs in California.

It should also be noted that the values for the period of the structure, and the period parameters were different between the IBC and CBC designs. The equation was similar, and would be the same for some structural systems, but not for all. Related to this, the factor R, which is concerned with the structural system was 3.5 for the IBC designs and 4.5 for the CBC design. The R factor is one of the most important in seismic design because it signifies the amount of inelastic action that will contribute to energy dissipation before yielding of the structural elements. Differences in this parameter will have significant impacts on design. The differences in the CBC and IBC regarding R can be explained since the factor is used in slightly different ways in the different methods. Also, considering inelastic action will change the period of the structure, which is why the period parameters for the methods also differed. Although they are small, differences like these are important, and can compound into a visible difference in design base shear.

5.2 *Seismic: Midwest Site*

This section details the analysis of the same low-rise structure in a site located in the Midwest (the intersection of 85° longitude and 40° latitude).

5.2.1 Design Assumptions

All of the basic assumptions—height, site class, structural system, standard occupancy, effective seismic weight—of the previous design were used again, except for the site location.

5.2.2 2003 IBC Design

The differences between this and the previous 2006 design lie with the mapped spectral acceleration, and the contours from the 2003 IBC are previously shown in Figure 4-4 ad Figure 4-6.

The short and 1-second mapped spectral accelerations were 19%g and 7%g, respectively. Again the maximum C_S controlled. Using the procedure and equations previously explained, the seismic response coefficient is 0.054 and the base shear is 163.1 kips. The results are shown in Table 5-5.

Table 5-5: 2003 IBC Midwest Seismic Design Results

h	45	ft	W	3000	Kip
R	3.5		S_{MS}	0.304	
I	1		S_{M1}	0.168	
C_t	0.028	ft	S_{DS}	0.203	
x	0.8		S_{D1}	0.112	
Site	D		T	0.588	s
S_s	19	%g	C_S	0.098	
S_1	7	%g	C_s min	0.010	
F_a	1.6		C_s max	0.054	
F_v	2.4		V	163.1	kip

5.2.3 2006 IBC Design

Again, the differences between the California and Midwest designs are in the mapped spectral acceleration. The contours of the relevant area can be seen in Figure 4-12 and Figure 4-14.

The short and 1-second mapped spectral accelerations were 20%g and 9%g, respectively. Like the California design, the maximum C_S controlled, most likely due to the small fundamental period. Using the procedure and equations previously explained, the seismic response coefficient is 0.070 and the base shear is 209.7 kips; results are shown in Table 5-6.

Table 5-6: 2006 IBC Midwest Seismic Design Results

h	45	ft	W	3000	kip
R	3.5		S_{MS}	0.320	
I	1		S_{M1}	0.216	
C_t	0.028	ft	S_{DS}	0.213	
x	0.8		S_{D1}	0.144	
Site	D		T	0.588	s
S_s	20	%g	C_S	0.104	
S_1	9	%g	C_s min	0.013	
F_a	1.6		C_s max	0.070	
F_v	2.4		V	209.7	kip

5.2.4 Comparisons and Discussion

Table 5-7 shows the results from the two designs side-by-side. Again, the bolded values highlight some important differences between the two designs.

Table 5-7: Midwest Seismic Design Comparisons

	2003 IBC	2006 IBC
h (ft)	45	45
R	3.5	3.5
I	1	1
C_t	0.028	0.028
x	0.8	0.8
Site Class	D	D
S_S (%g)	**19**	**20**
S_1 (%g)	**7**	**9**
F_a	1.6	1.6
F_v	2.4	2.4
T (sec)	0.588	0.588
W (kip)	3000	3000
S_{MS} (%g)	0.304	0.320
S_{M1} (%g)	0.168	0.216
S_{DS} (%g)	0.203	0.213
S_{D1} (%g)	0.112	0.144
C_s, value used	**0.054**	**0.070**
V, value used (kip)	**163.1**	**209.7**
% diff (from 2003 IBC)	-	**+28.6**

The difference between the 2003 and 2006 designs was an increase in the design base shear of about 28.6 percent. This is a large difference. The reason for the discrepancy between the two methods is the differences in the ground motion contour maps. The values read off of these maps are the basis for the construction of the response spectrum, and therefore the basis of the overall design of the building. Even if the values only change slightly, as show in the bolded values in Table 5-7, there can be large differences in the ending base shear values. Since the ground motion contour maps in the 2006 IBC show some changes from those in the 2003 IBC for numerous locations, these changes in design base shear values will be common. If numerous areas of the country will have to start designing structures to resist more lateral load, there will be significant impacts in design and construction, especially economically.

5.3 *Wind: California Site*

5.3.1 **Design Assumptions**

Since the largest wind-related code change is in California, the theoretical design site is located in the northern part of the state in the middle of the Sacramento Valley, as mentioned in the first part of this chapter.. The structure will be the same one as in the seismic design examples—low-rise, 45 feet tall and standard occupancy. Due to the relatively flat terrain and limited structures, in this part of the state, the usual assumption for wind loading is to use exposure category C.

5.3.2 **2001 CBC Design**

The standard occupancy leads to a wind importance factor of 1.0 (Table 4-3). Using Figure 4-2, the basic wind speed for the site is 75 mph. With this information the wind stagnation pressure, q_s, can be determined, (using interpolation) from Table 4-7 to be 14.5 psf. The pressure coefficient, C_q, is 1.4 (from Table 4-10: The last factor, C_e, is the combined height, exposure and gust factor is shown in Table 4-11. This factor is dependent on height. Once this factor is determined, Equation 4-6 is used to determine the wind pressure. The results, based on the height of the structure are shown in Table 5-8.

Table 5-8: 2006 IBC Wind Design Results

h (ft)	0-15	15-20	20-25	25-30	30-40	30-45
C_e	1.06	1.13	1.19	1.23	1.31	1.34
P (psf)	21.518	22.939	24.157	24.969	26.593	27.202

Similar to the IBC simplification, instead of designing the building in 5 or 10 feet vertical sections, the largest wind pressure would likely be applied to the entire structure. In that case, the design wind pressure would be 27.2 psf.

5.3.3 2006 IBC Design

While not all structures will qualify to be designed under Method 1, the simplified method, many low-rise structures will. Since wind loading does not tend to control the lateral loading of low-rise structures in seismically inclined areas, the simplified, probably more conservative, method will suffice.

The assumption of standard occupancy leads to a wind importance factor of 1.0 (Table 4-14). In addition, the topographic factor, K_{zt}, was assumed to be 1.0, which is the value for flat terrain, which is fitting for the site. Using Table 4-20, the building height adjustment factor, l, is 1.53. With Figure 4-9 the basic wind speed was determined to be 85mph. Along with the assumption that the roof slope is 25%, this value is used to determine the simplified design wind pressure, p_{S30}.

This method gives a design wind pressure for several sections of the building (shown in Figure 4-8). The results of these pressures are shown in Table 5-9.

Table 5-9: 2006 IBC Wind Design Results

Zone	A	B	C	D	E	F	G	H
p_{s30} (psf)	14.4	2.3	10.4	2.4	-6.4	-8.7	-4.6	-7
p_s (psf)	22.032	3.519	15.912	3.672	-9.792	-13.311	-7.038	-10.71

In order to simplify the design of the lateral force resisting system, an engineer would design the entire structure using one pressure instead of designing each section with a different structure. To be conservative, that single pressure would be the highest one, 22.0 psf.

5.3.4 Comparisons and Discussion

If the most conservative pressure value is used for both designs, the CBC design is about 5 psf, or about 20 percent, more conservative than the IBC method. A 20 percent difference is a large amount, and most likely due to the CBC design being a simplified, more conservative model than the IBC design. This has been acceptable in the past because predicted seismic design loads tend to exceed the design wind loads, even using the overly conservative CBC method. In addition to the quantitative differences between the codes, there are other disparities such as ease of use. The CBC method was exceedingly easier and straightforward, and the IBC method was more difficult and more time consuming. While this will be a negative short-term effect, it is not a justifiable reason to put off the code update since the IBC method is more accurate.

5.4 Summary of Seismic Differences

Concerning the design changes, the actual procedural change is not significant. In fact, the abandoning of the seismic zone (CBC) and category (IBC) method in favor of a method based on expected ground acceleration and response spectra is more accurate without being more difficult. The contour maps are small, and in some places hard to read, but there are on-line resources that will zoom in on the maps. The contour maps are based on 2500-year seismic events, but the spectral acceleration equations include a two-thirds factor that reduces the accelerations to 500-year events. This is neither mentioned nor explained in the code. The only advantage to this approach is that the two-thirds factor could be changed to change the design results without significantly changing any other part.

The results of the analysis done in sections 5.1 and 5.2 show that the impacts of the code update for most of the country, and the code change for California, will be felt everywhere. Some places throughout the country will find that the seismic design loads will increase, while California will generally find the opposite. This is a more significant change for some areas. In those areas, a large increase in the design base shear will greatly impact the necessary lateral resisting system in such a way that the member sizes will be larger, and more expensive. Despite the negative effects, the new earthquake design method in the 2006 IBC is more objective and more accurate, and therefore justified.

5.5 Summary of Wind Differences

Considered wind loads and the corresponding designs have to be such that the structural integrity and other important aspects are adequate throughout the design life of the structure in question. In general, standard design procedures are sufficient for common structures, but a more detailed method became necessary as building technologies advanced. For this reason building codes and standards, such as the ASCE building standard, has incorporated vast amounts advancing wind engineering into the methods. As a result the incorporation of this advanced knowledge, the codes have become "overly complicated and unwieldy" (Emil). In the case of the ASCE 7, even the simplified method is not very simple, especially since parts of the simplified method still refer to the even more difficult analytical method.

There is not a change between the 2003 and 2006 IBC, but the jump from the 2001 CBC wind loading method to that in the 2006 IBC is large. The new method will be less conservative, and therefore more accurate. Despite the increased accuracy in predicted loadings, the increased difficulty and new lengthy design process will not show widespread benefits in California. For many structures in the seismically active state, wind loadings do not dominate earthquake loads, but they must still be calculated. It is inefficient to spend such a large amount of time working through a convoluted design method that will not control. In addition, engineers with decades of experience are again rookies when it comes to new wind loading, so California is losing a great deal of expertise while the engineers learn and become comfortable with a new method.

6 Conclusions

6.1 Summary

The purpose of this thesis was to compare and contrast the 2001 California Building Code, the 2003 International Building Code and the 2006 International Building Code, and to evaluate the overall effects of the lateral code changes on both the design of structures and on the practicing engineers. To carry out this evaluation, design results from the three different codes were compared. This thesis considered three design scenarios for a simple low-rise structure: seismic loading in Southern California to compare the 2001 CBC, the 2003 IBC and the 2006 IBC, seismic loading in the Midwest to compare the 2003 IBC and the 2006 IBC, and wind loading in Northern California to compare the 2001 CBC and the 2006 IBC. The results of these calculations showed that the design base shear values from the 2006 IBC method are lower than the values from the 2001 CBC method for California locations, but higher than the values from the 2003 IBC for other areas, such as the Midwest. For the wind comparison, it was shown that California's adoption of the wind section of the 2006 IBC would result in a significant decrease of design wind pressures. These differences show that the adoption of the 2006 IBC will affect the lateral design load values, and therefore the design of a structure.

This study successfully showed what was intended, but there are limitations to carrying out the analysis in this way. First, only a couple of locations were analyzed. If other locations were considered, the analysis may have shown different, less significant, or opposite results. In addition, some of the differences shown in the seismic analysis were only a few percent. Differences this small are significant, but instead of resulting from true discrepancies in the codes, these small differences may be the possible error built into the procedures.

6.2 Conclusions

There are ramifications to code changes including differences in design values, as well as difficulties in learning new methods. With the code changes at hand, earthquake-prone areas of California will enjoy slightly relaxed seismic designs, and the whole state will have to learn and employ a more complicated wind design procedure. From the view of a design engineer in California, the new complicated wind procedure is the most worrisome of the changes. The rest of the country will endure higher design values for earthquake loading, but will not have to assume new design methods for either seismic or wind design loads.

As we acquire more knowledge of earthquakes through research or through experience, the seismic building code will inevitably change. The evolution of the code is a reflection of how well earthquakes are understood and how well the world is prepared to handle them. While updates are important to ensure the safety of structures, they can also come with negative consequences. When revising seismic code it is important to consider both the positive and negative ramifications. Is the increased ability for a structure to resist an earthquake worth it if the impacts of the change are high? And on the other hand, is avoiding change worth the structural damage, and even casualties, that could have otherwise been prevented? Although we do not necessarily know when or where, we do know that earthquakes will happen, and we can only hope that our structures have been built soundly enough to resist them.

The small benefits of protecting the wind-controlled structures in California from overly conservative designs, and of normalizing the wind loading design method throughout the country will not compensate for the negative impacts for engineers in California on the short term. However, in the long term, a unified national method will help the country as a whole, and hopefully the engineers in California will become accustom to using the new method. In addition, codes may again change and develop now that the country is standardized, and perhaps a simplified method, that is actually simple, will develop.

While the overall impacts may be large, the updated code will standardize and unify the whole country under one code that is more accurate than its predecessors. In addition, the IBC will have potential to expand beyond the boarders of the United States now that the entire country is using it. Also, the nature of the methods is such that all that is needed in order to apply the code elsewhere is a few maps detailing expected spectral accelerations and basic wind speeds. There are negative impacts, especially concerning wind loading in California, but this step toward a universal code will likely have positive ramifications.

6.3 Recommendations

To expand upon this study, it would be important to consider other locations to ensure that effects are uniform, or at least similar, throughout the country. Future research in this area might include a study of the effects of the code changes after they have happened. It would be useful to see how these changes actually impact the country after design engineers become accustom to the new code. In addition, it will be interesting to see what changes the next code cycle will bring since the IBC is updated every three years.

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