PERFORMANCE OF WOOD-FRAME CONSTRUCTION IN EARTHQUAKES

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SUMMARY

The performance of wood-frame construction is reviewed for some earthquakes in North America, New Zealand and Japan. The behaviour is related to peak ground acceleration (PGA). The conclusion is reached that the life-safety objective of building codes and various degrees of damage control have been met for single-storey houses for PGA of 0.6 g and sometimes higher. For multi-storey buildings life-safety has been met except where the soft-storey phenomenon was present. World-wide research is addressing this and other issues for improved quantification of seismic behaviour. It is recommended that in damage surveys more attention be paid to undamaged structures in the affected areas and that a comprehensive seismic instrumentation program for wood-frame structures be implemented.

INTRODUCTION

Wood-frame construction is by far the most common housing type in North America for single-family and low-rise multi-family dwellings. Most of these buildings are of the so-called platform frame construction, which is also gaining acceptance in other parts of the world. In the last few decades a large number of wood-frame houses have been subjected to earthquakes and this provides an opportunity to assess the general seismic performance of this type of construction. It is the purpose of this work to review the performance in some previous earthquakes and to relate the performance as much as possible to quantitative indicators of ground shaking intensity such as peak ground acceleration.

Elements of Wood-frame Construction.

The typical modern platform wood-frame house consists of a concrete foundation (also sometimes concrete block masonry), whereupon a platform is constructed of joists covered with plywood or oriented strand board (OSB) to form the floor of the ground level of the house. This platform is connected to the foundation with anchor bolts and on this base the walls are erected. The walls consist of a horizontal sill plate and nominally 2 inch by 4 inch (38 mm by 89 mm) or 2 inch by 6 inch (38 mm by 140 mm) verticals of one storey height at a spacing of typically 40 cm. Onto these verticals are nailed on the outside plywood or OSB, the inside spaces are filled with thermal insulation and then covered with a vapour barrier and gypsum board interior finish. The roof structure, generally consisting of prefabricated trusses, is attached to the top plate of the walls. Roofing consists of plywood or OSB sheathing nailed to the top chords of the trusses, frequently covered with asphalt or wood shingles. Windows and doors are inserted into the wall frames and electrical and plumbing is installed. The interior and exterior is then completed to various consumer preferences. See Fig. 1. Further details can be found in Ref. (4).

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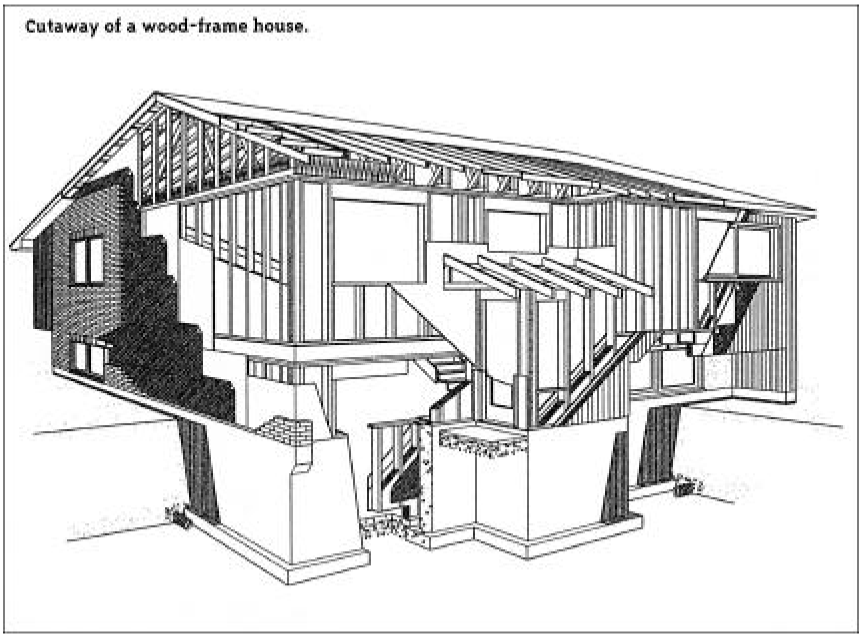

Fig. 1 Elements of Platform wood-frame construction. Source: Canada Mortgage and Housing Corp., Ref. (1).

Multi-storey platform wood-frame construction follows the same basic pattern as each succeeding floor is added on top of the previous one.

Another variation of wood-frame construction is the "balloon" type in which the vertical wall members are carried through the ceiling into the next storey. This is, however, much less commonly used than the platform type.

PARAMETERS THAT GOVERN SEISMIC RESPONSE OF BUILDINGS

Observations of number of earthquakes as well as theoretical considerations and experiments have shown the following parameters to govern the seismic response of buildings:

- the characteristics of ground movement at the building site, which can be characterized in the simplest manner by the peak ground acceleration (PGA). The PGA is particularly useful as an indicator of shaking intensity as it affects low-rise buildings, such as those considered here, since the PGA is largely governed by the high frequency components of ground motions and these affect directly the performance of the high-frequency structures of 1 to 4 storeys;

- the dynamic characteristics of the building, which include the natural modes of vibration, the natural frequencies (or their inverse, the natural periods) and the damping. These properties will determine how violently the building reacts to the ground motions to which it is being subjected; and

- the deformational characteristics of the building, such as stiffness, strength and ductility, which are interrelated and will affect the dynamic properties of the building. These properties are greatly affected by the building regulations that are followed in the building design and construction.

Building codes aim mainly at prevention of injury and death of people. This usually means avoidance of collapse, but at the same time the structure may be severely damaged during an earthquake and it may even have to be demolished. Thus under design objectives of life safety, cases where no deaths or serious injuries have occurred would be viewed as a “success”. If operational readiness or damage control was the design objective, however, the same severe damage would not represent a satisfactory outcome. The more stringent objectives carry cost implications, of course, but are expected to become more easily implemented with the introduction of “objective-based” building codes that are now being formulated.

Many building codes recognize two types of construction: 1) engineered construction and 2) construction by conventional rules. Most single-storey houses and many 2 to 4 storey wood-frame buildings are built according to the conventional rules of construction.

BEHAVIOUR OF WOOD-FRAME CONSTRUCTION IN PREVIOUS EARTHQUAKES

General Approach. As can be appreciated from the previous discussion of factors that affect the seismic performance of buildings, the process is a complex one and difficult to analyze, especially for wood-frame construction since a large number of diverse components are involved. Fortunately, many of the earthquakes in the last 30 to 40 years have yielded a significant amount of instrumental data on how the ground had moved and how some buildings responded, along with damage surveys. From the seismograph data, contour lines of PGA can be drawn to represent equal intensity of shaking; this is employed here. Because instrumented ground motion stations are still relatively sparsely distributed, however, drawing contours through a small data set must be recognized as being approximate since even over small distances some variation of ground motions can occur. On a statistical basis, however, this procedure can be expected to provide useful information concerning dominant trends.

Since this survey confines itself to wood-frame construction, only those earthquakes will be examined where these types of buildings were particularly affected: Alaska Earthquake, 1964; San Fernando Earthquake, California, 1971; Edgecumbe Earthquake, New Zealand, 1987; Saguenay Earthquake, Quebec, 1988; Loma Prieta Earthquake, California, 1989; Northridge Earthquake, California, 1994; Hyogo-ken Nanbu Earthquake, Kobe, Japan, 1995.

Fig. 2. House in foreground suffered only minor damage though being dropped into an earth slide area. Alaska Earthquake, 1964. Source: Forest Products Laboratory, Madison, WI. US Dept. of Agriculture, 1964.

Alaska Earthquake, 1964. With magnitude M8.4 this was one of the largest earthquakes in North America in this century and was characterized by intense ground shaking that caused major landslides in the populated area of Anchorage (Ref. (11)). Both ground settlement and large horizontal and vertical ground displacements caused significant damage to foundations and to the structures supported thereon, resulting in a large number of collapses of buildings and deaths. Unfortunately, no recordings of ground motions were obtained. The performance of wood-frame houses was punctuated by the structural integrity of buildings that literally slid down failed slopes or were otherwise subjected to large ground movements. The box-like wood-frame structures stayed largely intact and "rode out" the earthquake, often coming to rest in an unusable orientation for immediate occupancy! (Fig.2).

San Fernando Earthquake, California, 1971. This earthquake of magnitude M6.7 occurred in a northerly suburban area of Los Angeles and consequently affected a large number of single-family homes as well as hospitals and commercial buildings and resulted in 64 deaths. The unreinforced masonry four- and five-storey Veterans Hospital collapsed, killing 46 occupants. Other hospital and office buildings built of reinforced concrete collapsed, or were severely damaged to the point where they had to be demolished, as for example the Olive View Hospital. From a contour map drawn from recorded peak ground accelerations (Ref. (12)) it may be observed that the residential areas of San Fernando experienced peak horizontal ground accelerations of 0.6 of gravity (g) and greater.

Older wooden houses in the San Fernando area suffered damage ranging from minor to partial collapse. Newer two-storey apartment buildings with large ground-level openings were also severely affected (see Fig. 3). These damage cases can be categorized as follows:

Damage	Deficiency
Houses sliding off the foundations	Frame not bolted to foundation
Collapse of "cripple walls" in crawl space	No lateral bracing
Collapse of add-ons such as porches	Inadequate lateral strength and ties
Collapse of masonry chimneys	Incompatibility of stiffness
Major distortion of weak first storey	Large openings, inadequate strength

The importance of such deficiencies in residential construction was first widely recognized in this earthquake and many of these weaknesses have since then been rectified. Unfortunately, not all these lessons have been readily converted into practice and it took other earthquakes, notably the one at nearby Northridge, to re-emphasize the importance of these shortcomings in the seismic performance of wood-frame construction.

Aside from these specific types of failures, the majority of the then-modern wooden houses performed well, especially when the life-safety criterion is considered. Even where the building slid off the foundation or the cripple wall collapsed, the remainder of the building stayed intact and prevented serious injury or death to occupants. The same general observation can be made where the chimney collapsed; in most cases the remainder of the house suffered no appreciable damage. A weakness in ground floors in multi-storey buildings due to large openings from garage doors or open parking was also highlighted in this earthquake. A more detailed damage survey is presented in Ref. (10).

Edgecumbe Earthquake, New Zealand, 1987. This event on the North Island consisted of a main shock of magnitude M6.3, preceded by 7 minutes by a magnitude 5.2 fore-shock, and followed by four aftershocks with magnitudes greater than 5. These quakes were centred in a rural area and small towns. The only ground motion record obtained came from the base of the Matahina Dam, over 20 km from the epicentre of the main shock, with a peak horizontal acceleration of 0.32 g. The town of Edgecumbe (population 2000) may have experienced greater accelerations since it is located only about 8 km from the epicentre of the main shock. Considerable damage resulted in all types of structures - railways, bridges, residential and industrial buildings, storage tanks, electrical equipment and municipal services. Widespread liquefaction of the ground was observed in a 10-km radius around Edgecumbe. No deaths or serious injuries were reported. See Ref. (8).

The wood frame houses in this area were typically built on concrete strip or concrete block foundations, with a shallow crawl space below the ground floor. Many of the walls were lined on the interior with gypsum board and covered on the outside with a brick veneer. No exterior sheathing was used, although the wood frame incorporated K-bracing or diagonal bracing members.

Of the approximately 6500 houses in the affected region, fewer than 50 suffered substantial damage but none collapsed, Ref. (8). Damage consisted of houses sliding off the foundation, cracking and collapse of the brick veneer on the building exterior, collapse of chimneys and failure of foundation posts and roof struts. The seismic weaknesses exhibited here follow those observed elsewhere, namely lack of foundation connection or bracing, incompatibility of brittle and heavy materials (i.e. masonry) with wood construction, and lack of lateral bracing.

Saguenay Earthquake, Quebec, 1988. Although the Saguenay Earthquake in northern Quebec, Canada, with magnitude M5.7 was not as powerful as some of the others surveyed here, it was the largest earthquake in the last 50 years in eastern North America. The epicentre was located in a lightly populated area 150 km north of Quebec City. Peak ground accelerations in the population centres of Chicoutimi, 36 km from the epicentre, and on the north shore of the St. Lawrence River, 100 km away, were of the order of 0.12 to 0.17 g. See Ref. (7). The ground motions were of a high frequency type and due to the low seismic attenuation rates in the earth's crust in eastern North America, the ground motions were felt at distances of 500 km and beyond.

Most of the damage to wood-frame buildings up to 2 storeys was limited to cracks in chimneys, foundations and brick veneer walls. No cases of near-collapse or deaths were reported. The damage pattern correlated strongly with the presence of soft soil deposits and damage in the houses was attributed mainly to foundation soil displacements rather than structural weaknesses in the superstructure (Ref. (7)).

Loma Prieta Earthquake, California, 1989. This earthquake with magnitude M7.1 had its epicentre 100 km south of San Francisco, but its effects reached well beyond to Oakland on the north shore of San Francisco Bay. Total casualties were 62 deaths and over 3000 injured; alone 49 persons died in the collapse of the double-deck freeway in Oakland. This earthquake caused considerable damage and some collapses of older unreinforced masonry buildings near the epicentre and as far away as Oakland. A contour map constructed from a relatively sparse set of recorded peak ground accelerations is presented in Ref. (9).

Fig. 3 Failure of supports due to weak first storey. San Fernando Earthquake, 1971.

Fig. 4. Almost collapsed wood-frame apartment building, Marina District, San Francisco. Loma Prieta Earthquake, 1989.

Fig. 5. Collapsed first floor parking area, Northridge Earthquake 1994.

A number of older four-storey wooden apartment buildings located on fill materials in the Marina Bay district of San Francisco collapsed onto the ground floor that consisted entirely of garage openings and therefore represented a very weak first storey. The upper 3 storeys, however, stayed intact. Some other buildings of this type nearly collapsed (see Fig. 4). Ground motion amplification of the soft soils at that site also contributed to this damage pattern. Further details can be found in Ref. (5).

Wood-frame houses located in and near the epicentral region survived the shaking mostly with repairable damage. Some of these houses near the epicentre were likely subjected to peak ground accelerations as large as 0.5 g and possibly larger.

Northridge Earthquake, California, 1994. The Northridge Earthquake with magnitude M6.7 was notable for its high ground accelerations, both horizontally and vertically, and in places exceed the acceleration of gravity (1.0 g). A contour map of peak horizontal ground accelerations is shown in Ref. (3). The earthquake caused extensive damage to residential, institutional, and commercial buildings and to the highway and freeway system of this area about 20-km north-west of Los Angeles. This represented the most intense ground shaking that had so far been recorded in a populated area in North America. Over 30 persons were killed and estimates of property losses ranged from US$ 30 - 40 billion.

This earthquake again drew attention to a weakness that had already been recognized, practically next door, in the 1971 San Fernando earthquake - the weak first storey in multi-storey wood-frame apartment buildings. Whereas in the 1971 earthquake these weak buildings merely distorted severely but remained standing, in this larger 1994 earthquake a number of ground floors collapsed (see Fig. 5). In one apartment complex 16 occupants were killed. Such collapses are perhaps not surprising when one considers that the horizontal ground accelerations, combined with the vertical accelerations of comparable amplitude, exceeded the nominal horizontal design acceleration of 0.4 g by factors of 2 and more. Four deaths also occurred in three single-family houses that slid down a hillside and collapsed.

Despite these tragic failures, other wood frame construction performed exceedingly well. As in other earthquakes, chimneys were severely damaged, while the rest of the building survived without significant problems. In a statistical study of the seismic performance of residential construction in the Northridge earthquake (Ref. (6)) the authors state:

> "SFD [Single family dwelling] homes suffered minimal structural damage to elements that are critical to the safety of occupants. Structural damage was most common in the foundation system. The small percentage of surveyed homes (approximately two percent) that experienced significant foundation damage were located in areas that endured localised ground effects or problems associated with hillside sites."

Hyogo-ken Nanbu Earthquake, Kobe, Japan, 1995. The earthquake that hit the city of Kobe in the Hanshin area of Japan on January 17, 1995 was so far the most damaging earthquake in modern times, with estimated losses of well over US$ 100 billion and loss of life of nearly 6 000 inhabitants. Its magnitude was 7.2 on the scale of the Japanese Meteorological Agency, M6.8 on the Richter scale. The area had not anticipated an earthquake of this magnitude and therefore building design and construction was ill-prepared. Extensive damage and major collapses occurred in all types of structures - elevated highways, bridges, port facilities, utility services, rail and subway installations, and low and high-rise buildings of concrete, masonry, steel and wood (Ref. (3)). Among the wide-spread destruction there were also numerous recently constructed buildings that survived the earthquake, some with no visible damage.

While the epicentre of the earthquake was located on the nearby island of Awaji southwest of Kobe, the rupture zone extended through the centre of the city to its neighbour city Nishinomiya to the east. Peak ground acceleration as high as 0.8 g were recorded in densely populated areas; a large part of southern Kobe and the cities of Ashiya, Nishinomiya and Takarazuka experienced over 0.6 g peak ground acceleration. These horizontal motions were also accompanied by large vertical accelerations, often as large and sometimes larger than the horizontal ones. This phenomenon is similar to that observed in the Northridge earthquake of 1994 and appears to be characteristic of ground shaking in the epicentral regions of an earthquake.

The wooden houses hardest hit were those constructed immediately after World War II (Ref. (2)). These houses consist of post-and-beam construction, with walls formed by horizontal boards nailed to the uprights, in-filled with bamboo webbing and covered with clay. Exterior finishing is typically stucco. Heavy roofs built of burnt clay tiles proved disastrous in the earthquake. Within the 0.6 g contour of PGA a majority of this type of building collapsed or was heavily damaged. This gave rise to devastating fires from broken gas lines and stoves; the interruption of the water supply greatly aggravated the situation.

Among the sea of devastation of older style houses were examples of modern wood construction and wood-frame houses that showed no visible signs of distress (Ref. (2)). These houses were located in areas of severe shaking and demonstrate that modern wood-frame construction can withstand earthquakes with peak ground accelerations of 0.6 g or more with little or no damage.

The wood-frame demonstration projects popularly known as Seattle-Vancouver village in the Nishi-Ward of Kobe city and the Hankyu Nishinomiya Housing Park contained modern wood housing, the former only wood-frame construction, the latter both post-and-beam and wood frame types. Both these areas were severely shaken, the Vancouver-Seattle village in the neighbourhood of 0.3 to 0.5 g, the Hankyu Nishinomiya Housing Park higher than that. The wooden houses in both of these projects showed no visible damage, except for some roof tile breakage (Ref. (5)).

ASSESSMENT OF PERFORMANCE

On the basis of observations of the performance of wood-frame buildings in previous earthquakes the following assessment is made:

- Single-storey wood-frame houses have performed well from a life-safety perspective when subjected to PGA of 0.6 g and even higher. Their performance has demonstrated that the life-safety objective inherent in building codes has been satisfied. In fact, for most of the houses the damage sustained is of a minor nature, showing that the objective of damage control has also largely been achieved.

- Two-storey wood-frame buildings met the life-safety criterion, although some were seriously damaged in areas of 0.6 to 0.8 g PGA.

- For three and four-storey wood-frame structures the life safety objective has also largely been achieved for PGA of 0.6 g and larger, with some exceptions. The exceptions generally were cases of weak first storeys, where the resistance or strength was clearly inadequate and could not meet the loading demands of the earthquake.

IMPLICATIONS FOR RESEARCH AND DESIGN

From the above damage survey it follows that if the same good performance is expected for multi-storey wood-frame buildings as was demonstrated for the single-storey ones, then the weak first storey phenomenon has to be addressed. This appears achievable by proper design for engineered buildings and, for conventional construction, by specifying minimum wall dimensions and anchorage requirements in building regulations and codes and standards, and then enforcing them. To provide the quantitative information needed and improve predictability of performance, a number of multi-faceted research programs into seismic behaviour of wood-frame construction have been launched by the following organizations: Building Research Institute (BRI) of Japan; BRANZ in New Zealand; Forintek Canada Corporation in Canada; and the Wood-frame Project of the California Universities for Research in Earthquake Engineering (CUREe). Many other institutions also carry out research into one or more aspects of seismic behaviour of wood-frame construction: behaviour of joints and wall elements, mathematical modelling, component and full-scale testing, and development of design aids and codes and standards. Earthquake damage surveys also provide valuable data for further evaluation. It is, recommended that more attention be paid to undamaged structures in the affected areas so that we can also learn from what is done right and not only from what was done wrong! Detailed investigations are, however, hampered by a lack of strong-motion records of the seismic response of wood-frame buildings. Such records would be invaluable for the further quantification of the seismic response of this common type of construction.

SUMMARY AND CONCLUSIONS

An examination of the seismic behaviour of wood-frame construction in a number of recent earthquakes has shown that the life-safety objective of building codes has largely been satisfied for PGA of 0.6 g and sometimes higher, except for buildings where the soft-storey phenomenon was present. Furthermore, various degrees of damage control have also been demonstrated.

Research efforts are under way in many parts of the world in order to further improve the life-safety and damage control of wood-frame construction in earthquakes. It is recommended that a comprehensive seismic instrumentation program for wood-frame structures be implemented so that the actual seismic performance of these common types of structures can be observed and better quantified.

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