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SEISMIC BEHAVIOUR OF RESIDENTIAL WOOD-FRAME CONSTRUCTION IN BRITISH COLUMBIA: PART II – PERFORMANCE REQUIRMENTS

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ABSTRACT

During the last two decades, significant earthquakes such at the 1989 Loma Prieta and the 1994 Northridge earthquakes in California and the 1995 Kobe earthquake in Japan, have demonstrated that seismic hazards pose a credible threat to residential wood-frame construction. Lessons learned from these earthquakes help us understand now that many new or older houses in British Columbia are at a significant seismic risk because of excessively large openings or deficient sheathing material.

This paper presents a simple method to evaluate the performance of existing wood-frame buildings, and provides guidance on required strength levels to meet code-compliant performance objectives. Strength values are provided to account for traditionally non-engineering materials such as stucco, gypsum wallboard and horizontal board sheathing. Performance curves are provided to allow for a displacement-based design of a typical wood-frame house located in Vancouver, British Columbia on wide range of soil types.

Introduction

Single family dwellings constitute 56% of all residential dwellings in British Columbia, with the vast majority of these being of wood-frame construction (Ventura, et al. 2005). New wood-frame construction in Canada is governed by the National Building Code of Canada (NBCC), which specifies provisions for seismic design, for buildings of all sizes. Under the latest 2005 edition of the NBCC (2005, NBCC), new single family wood-frame construction should be specifically designed to resist lateral shaking due to earthquakes.

However, the 2005 NBCC is relatively new. Most of the residential stock was designed using the 1990 NBCC or older codes. Smaller buildings, including most single family dwellings, fell under Part 9 of the 1990 NBCC. Part 9, Housing and Small Buildings, provided prescriptive requirements for design, which did not specifically address seismic hazards, as it did in Part 4, Structural Design, for larger structures. As a consequence many existing single family wood-frame dwellings have an unknown seismic performance, which is possibly below the safety standards set for engineered buildings.

While there have not been any major seismic events in the Lower Mainland of British Columbia in recent

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history, 2.8 million British Columbians live in a region of high seismicity. The 1994 Northridge earthquake in California was responsible for 24 fatalities, $20 billion dollars in property damage and the loss of approximately 48,000 housing units, all due to the failure of wood-frame residential construction (CUREe, 2002). There is clearly a need to ensure that the residential wood-frame buildings in British Columbia have adequate seismic performance to mitigate the damage from a major earthquake.

This paper presents the results and recommendations of a study on the Seismic Performance of Residential Wood-frame Construction in BC. Using advanced analysis techniques and state-of-the-art experimental data, performance curves for four types of residential construction in British Columbia are developed using seismic hazard data and drift limits from the 2005 National Building Code of Canada. The details of the technical development of the performance curves are presented in a companion paper entitled "Seismic Behaviour of Residential Wood-frame Construction in British Columbia: Part I – Modeling and Validation" (White and Ventura, 2007), which also summarizes the experimental data that was the basis of the modeling.

The reports associated with this study (EERF Reports No. 06-02 and 06-03, 2006) are available from the Canadian Mortgage and Housing Corporation.

Objectives

The objective of this study was to build a set of performance recommendations for wood-frame residential construction in BC, by making use of the latest technology and research available. The following objectives were met to ensure a high level of accuracy in this performance-based approach:

- The contributions of stucco and gypsum wall board, which have been traditionally considered "non-engineering" materials, have been quantified.
- The non-linear shearwall models were developed based on local experimental tests of wall systems.
- The full non-linear house models were validated against full-scale shake table tests of house specimens built to local construction standards.
- Local seismic and soil conditions were accounted for in the analysis.
- All computer programs and/or analysis methods used in this study were independent and already peer reviewed.

Applicability

The tables and performance curves presented in this paper provide a simple and rational method for the assessment of existing or the design of new buildings small residential wood-frame buildings in British Columbia. This material is intended to compliment the experience and judgment of engineers, architects and/or contractors, by providing guidance on the non-linear behaviour of small residential wood-structures during an earthquake.

The FEMA-440 Displacment Coefficient Method, a non-linear static procedure, was used to develop the performance curves. This method, which is based on numerous non-linear dynamic analyses, allows one to estimate the displacement demand of structure, accounting for the non-linear characteristics of the structure (i.e. backbone curve and hysteretic properties), using an acceleration spectrum. The details of the implementation of this method are given in White and Ventura (2007).

Four types of residential construction are considered in this study. These are all single family two-storey dwellings of light-frame wood construction. All have gypsum wall-board interior finish, but have different sheathing materials. Seismic motion is resisted by shearwalls, which are comprised of both the interior finish and sheathing materials.

The four types of the houses are defined by the composition of their shear walls:

1) Blocked plywood/OSB Shear walls with exterior stucco cladding and gypsum wall board interior finish.
2) Blocked plywood/OSB shear walls with gypsum wall board interior finish.
3) Unblocked plywood/OSB shear walls with gypsum wall board interior finish.
4) Horizontal board sheathing (shiplap) with gypsum wall board interior finish.

For more details, including drawings, of the houses that were used to develop the results of this study, see EERF Report No. 06-03 (2006).

While this study was based on experimental test and analyses of specific buildings, the results can be applied to similar structures. The following three sections list some of the assumptions and limitations of this study.

Connections and Load-path

For any wood-frame structure to resist lateral movement there must be proper connections between the structural walls, diaphragms and foundation, such that a continuous load-path can be drawn from the roof to the ground. Additionally the "weak-link" in this chain of structural elements or components must be the shearwalls, as they are more ductile and are not prone to brittle failure.

Connections to the diaphragms and anchors to the foundation must be designed to be significantly stronger than the shear walls, to ensure that a brittle failure does not occur. See Engineering Design in Wood, O86 -2001 (CSA, 2001) or the Residential Guide to Earthquake Resistance (CMHC, 1998) for more information on connection detailing in seismic areas.

Construction Irregularities

The results of this study are based on two storey structures with no significant vertical or plan irregularities. The top and bottom storey have the same floor plan and the same lateral strength. The walls are relatively closely spaced, and no single wall has an excessive amount of openings. As such there are no significant eccentricity effects.

While ideally the results of this study should only be applied to similar structures, they can also be applied to structures that do not have major irregularities. These results should be suitable for 1 to 3 storey structures, where the majority of the deformations occur in the 1st floor. If the upper floors have significantly less capacity than the 1st floor, these results may be invalid. Structures with significant plan eccentricities and flexible diaphragms should treat each wall system as independent. Each of these wall systems should have enough capacity to resist the seismic forces associated with its tributary weight. If this cannot be done, then it is recommended that the structure in question be assessed or designed by a qualified structural engineer.

Local Site Conditions

While the study takes into account the variation in seismic demand on the five site classes (including soil structure interaction on Site Class E), it does not account for more localized site conditions. If the structure is built on or near a slope, or if there is a chance of liquefaction, a geotechnical engineer should be consulted.

Seismic Demands

The seismic demands used in this study are based on the provisions of Section 4.1.8 of the 2005 NBCC. These demands represent a major ground motion with a 2% in 50 year probability of exceedance. The

2005 NBCC details a methodology for constructing acceleration design spectrum for many locations in Canada and for the five different soil types (site class). Fig. 1 shows the seismic demands for Vancouver, British Columbia.

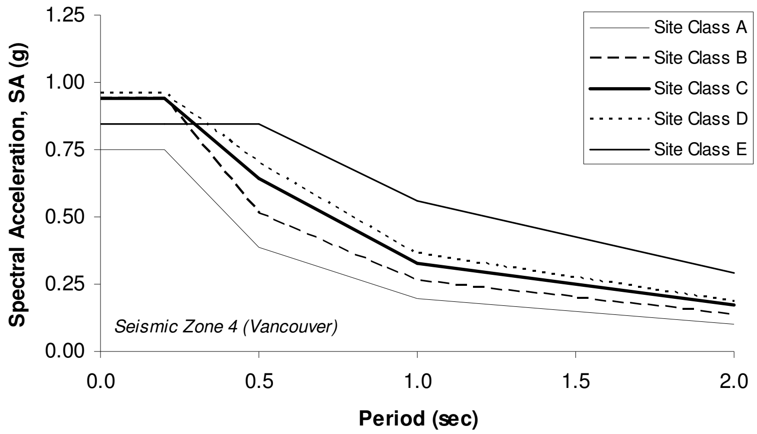

Figure 1. 2005 NBCC Seismic Demands for Vancouver, British Columbia with 5% Damping.

Seismic Zones

The province was divided into seven seismic zones to allow for a more reasonable number of performance curves. Each of these seismic zones was defined by the seismic hazard data for a typical municipality within the zone. The five zones relevant to this study and their representative municipality are Zone 2 (Princeton), Zone 3 (Chilliwack), Zone 4 (Vancouver) and Zone 5 (Victoria). Fig. 2 shows a portion of the Seismic Zone Map used in this study. For further details of the seismic zones see EERF Report No. 06-02 (2006).

Soil Type (Site Class)

The seismic demands are sensitive to the soil type (site class) on which the structure is built upon. See Section 4.1.8.4 of the 2005 NBCC for details on how the site class is determined. The acceleration spectra in Fig. 1 demonstrate the differences in seismic demands for each of the site classes. Soil-structure interaction has also been incorporated into the performance curves for Site Class E results. The soil-structure interaction method was taken from FEMA-440 (ATC, 2005), and details of its implementation are discussed in the EERF Report No. 06-03 (2006).

Fig. 3 shows a portion of the Soil Hazard Map for the Lower Mainland of British Columbia. See EERF Report No. 06-02 (2006) for more details.

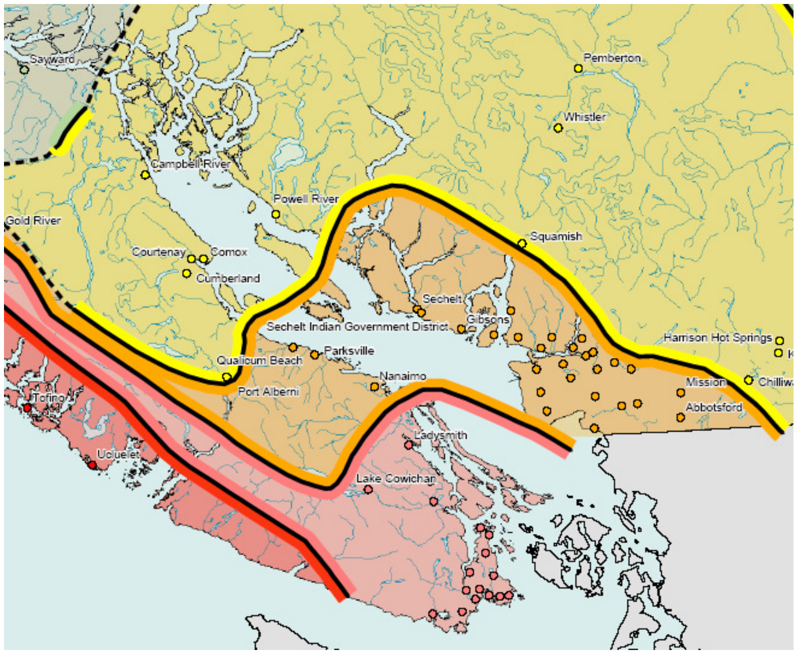

Figure 2. Seismic Zones in South-western British Columbia.

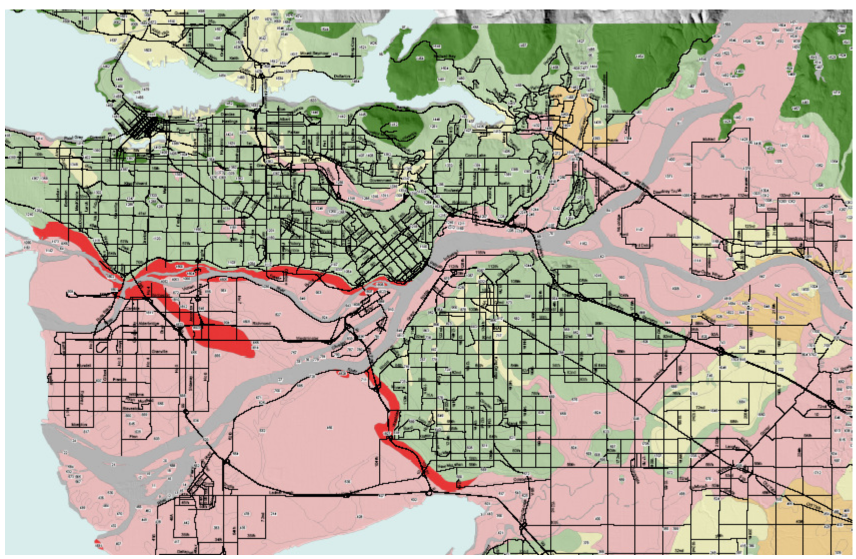

Figure 3. Soil Hazard Map for the Lower Mainland of British Columbia.

Seismic Performance

Damage in wood-frame structures is readily related to the maximum interstorey drift (i.e. lateral deformation at the top of a storey, relative to the bottom of the storey, divided by the storey height). "Performance" is used in this case to reflect the damage (or drift) of a given wood-frame structure to a prescribed seismic hazard level.

Fig. 4 shows performance curves for a Type 3 house in Seismic Zone 4 (Vancouver). These curves plot the base shear strength of the wood-frame structure versus the maximum interstorey drift (as a percentage of the storey height) that would occur during a design level earthquake. Base shear strengths have been normalized by the weight of the house (*W*) and expressed as a percent. *W* is defined as the total weight of the building above the mid-height of the first storey of the house, including 25% of the snow load (see Section 4.1.8 of the 2005 NBCC). In EERF Report No. 06-02 (2006) there is a set of performance curves for each house type (1 to 4) in each of the seismic zones (2 to 6). Each plot shows curves for the five site classes (A to E), ranging from maximum interstorey drifts of 1% to 4%.

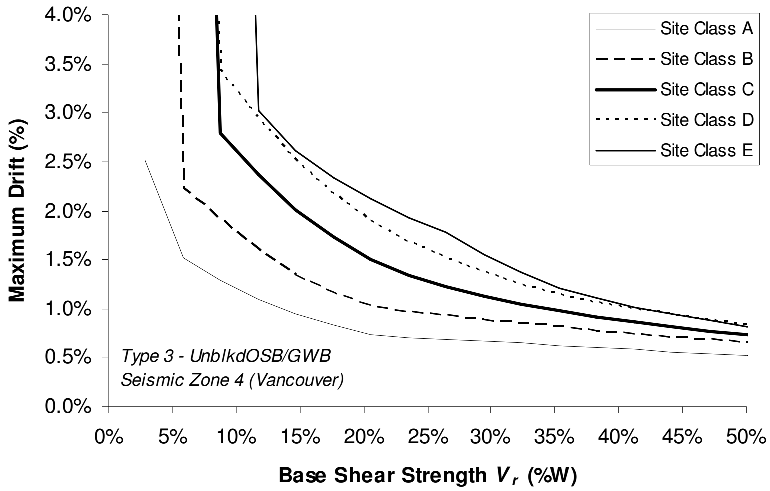

Figure 4. Seismic Performance Curves for Type 3 House in Vancouver, BC.

The performance curves can be used to determine the base shear strength demands required to limit the drift (damage) in a house, or conversely to estimate the drifts that would occur in an existing house. To determine the required strength, follow this procedure:

1) Identify what Seismic Zone and Site Class the house is located on (see EERF Report 06-02, 2006).
2) From an inspection of the house, determine which house type most closely matches the composition of the shearwalls.
3) Determine what performance level (maximum drift) is required (see below).
4) Find the appropriate table from EERF Report 06-02 (2006) that corresponds to the house type and Seismic Zone.
5) Using the correct line for the Site Class, determine the required base shear strength at the drift level from step 3) above.

To find and estimate of the expected drift levels in a design level ground motion, follow the procedure above, but in step 3) calculate the base shear strength of the structure (see below) and in step 5) determine the drift from the strength calculated.

Performance Level (Drift Limits)

The 2005 National Building Code of Canada (2005, NBCC) suggests a maximum drift of 2.5%. This corresponds to a life-safety performance objective, where there are no fatalities and the occupants can safely exit the building. Base shear capacities corresponding to the 2.5% drift in Vancouver, BC are listed on Table 1.

Table 1. Shear Strength Requirements for Seismic Zone 4 (Vancouver).

Minimum Shear Strength (V_r) Requirements (%*W*)				
Site Class	*Type 1*	*Type 2*	*Type 3*	*Type 4*
A	3%	3%	3%	5%
B	5%	6%	6%	9%
C	11%	9%	11%	14%
D	15%	12%	15%	19%
E	20%	14%	16%	19%

Some sources (APEGBC, 2006) suggest that drift limits of up to 4.0%, for light wood-frame construction, are still within life-safety performance levels. If the structure is of high importance, a higher performance objective, such as immediate occupancy or damage mitigation might be required. 2005 NBCC requires a maximum 1% drift for post-disaster structures. These performance curves show the relationship between base shear capacity and maximum drift. The maximum drift is governed by the load bearing walls, which must limit their displacement to prevent a P-Delta failure. Non-load bearing walls can tolerate higher drifts; however, most small residential structures without high eccentricities will see the same drift in all walls. Users of the tables will have to decide what level of drift is required to meet the desired performance objective.

Calculating Base Shear Strength (V_r)

The performance curves can be used for both new and existing structures. However these curves are inappropriate for houses that have a deficiency in their load path, which should be assessed by a qualified structural engineer. Suggested factored strengths for new and existing sheathing materials are listed below in Table 2. Strength values on this table come from APEGBC (2006) except stucco sheathing, which was based on Taylor et al. (2003).

Table 2. Suggested Factored Shear Strength, v_r, for Wood-frame Construction.

Sheathing or Finish Material	Suggested Factored Strength
Blocked OSB or Plywood	See O86-2001
Unblocked OSB or Plywood	3.5 kN/m
Horizontal Shiplap	0.6 kN/m
Diagonal Shiplap *(House Type 3)*	6.0 kN/m
Gypsum Wallboard	1.1 kN/m (per side)
Nailed Stucco with Welded Wire Mesh	5.5 kN/m

Notes: To qualify as diagonal shiplap, the boards must be inclined at 30 degrees or more from the horizontal. The values for gypsum wallboard can be used for a plaster interior finish as well.

To calculate the factored base shear strength (V_r) of an existing or new house follow these steps:

1) Determine what materials are used in the walls of the building, and find them on Table 2.
2) Take the length of wall covered by each material and subtract the length of any openings (i.e. where the wall is not continuous from the floor to the ceiling).
3) With the exception of Blocked OSB/Plywood, multiply the length (without openings) by the factored strength per meter, for each material on each wall, by the values shown on Table 2, to obtain the v_r of that wall or wall component.
4) For Blocked OSB/Plywood use the Wood Design Manual O86 -2001 (CSA, 2001), to determine the factored shear strength (v_r) of each shear wall or shear wall component.
5) Sum the factored shear resistances of all the walls and wall components, $\sum v_r$, (in one of the orthogonal directions of the building) to obtain the factored base shear resistance of the entire

structure, V_r. Repeat for the other direction of the building.

6) Do not use the R_o and R_d values listed in the 2005 NBCC. They have been accounted for already. See below.

Compare the strength (in %W) to those obtained from the performance curves. If the calculated strength is greater or equal to the number on the curve, then the house will achieve the desired performance level (drift). If the strength is less, than it requires a seismic retrofit. Make sure to check both orthogonal directions.

Ductility and Overstrength Factors

The performance curves already take into account the code specified R_d and R_o factors. R_d is implicit in the modeling and analysis, and R_o has been divided out of the final base shear (V_r) numbers given in the figures. House Types 1, 2 and 3, use an R_o of 1.7. House Type 4 has an R_o of 1.0.

Example 2-storey Wood-frame House in Vancouver, BC

Fig. 5 illustrates a simplified plan view of the ground floor of a 2-storey house in Vancouver, BC, located on Site Class C. The exterior walls are sheathed with unblocked plywood. All of the interior walls have a gypsum wall board finish. The 2nd storey has a very similar layout and construction, and does not need to be considered separately. The weight of the house, above the mid-height of the first storey is 186 kN, which includes 25% snow load from 2005 NBCC.

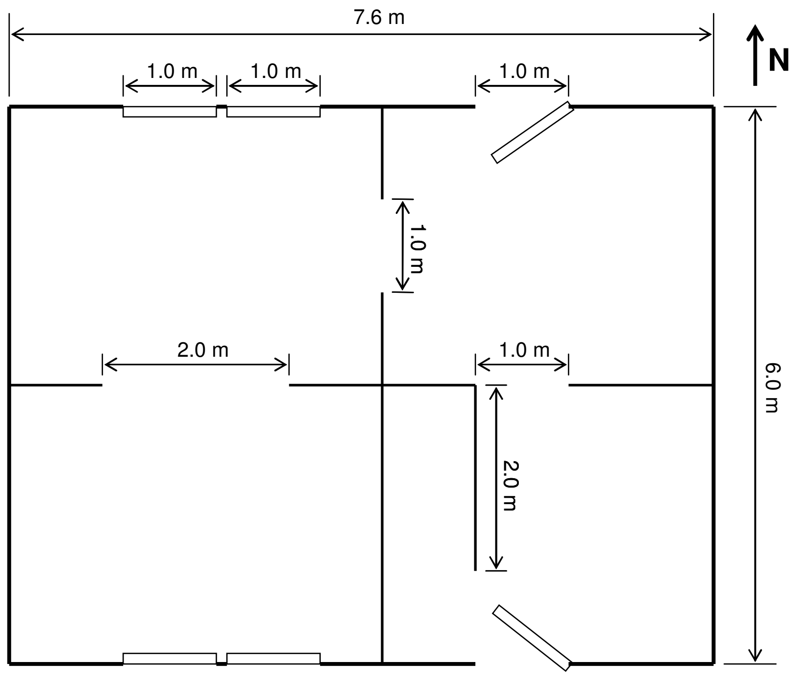

Figure 5. Floor Plan of a “Typical” 2-storey Vancouver Wood-frame House.

For this purposes of this example only the East-West direction will be considered. However, both directions need to be considered for any assessment or retrofit.

The shear resistance, v_r, from each material type is calculated as follows:

Unblocked Plywood has a capacity of 3.5 kN/m (Table 2). The total length of the walls (minus openings) with plywood is:

7.6 m – 3m (North facing wall) + 7.6 – 3m (South facing wall) = 9.2 m

$v_{r\text{-}plywood}$ = 9.2 m × 3.5 kN/m = 32.2 kN

Gypsum wallboard has a capacity of 1.1 kN/m/side (Table 1). The total length of the walls (minus openings) with gypsum wall board is:

2 × [7.6 m – 3 m] (Exterior) + 2 × [7.6 m – 3 m] (Interior 2 sides) = 18.4 m

$v_{r\text{-}gwb}$ = 18.4 m × 1.1 kN/m = 20.2 kN

The total factored base shear strength of this house is:

$V_r = \sum v_r$

V_r = 32.2 kN + 20.2 kN = 52.4 kN

The base shear expressed as a percentage of the weight of the building is:

V_r = 52.4 kN ÷ 186 kN = 28%*W*

The Seismic Hazard Map (EERF Report No. 06-03, 2006) indicates that Vancouver is in Seismic Zone 4. The construction materials used in the shearwalls of this house most closely match the Type 3 house. Table 1 indicates that a Type 3 house located on Site Class C requires a factored base shear strength of 11%*W*. Since this is less than the base shear resistance (28%*W*) the example house is well within acceptable performance levels, and does not need to be retrofitted, unless there are problems with the load-path (connections to diaphragms and or foundation).

Retrofitting Structures

While the house in the above example performed very well, it is not typical of deficient construction. Some at risk wood-frame dwellings are constructed on poor soil (Site Class E), have horizontal shiplap sheathing instead of plywood or OSB, have larger percentage of openings, and the interior walls are not properly connected to the foundation or diaphragms.

If the capacity of the building is below the demand, the capacity must be increased. Simply develop a retrofit scheme, and check it again using the tables provided. Be cautioned that the house type may change depending on the materials used in the shearwalls.

Conclusions

This paper demonstrates a performance-based method for determining the strength requirements of shearwalls of wood-frame residential construction of BC. However it only covers a limited number of building types. While it is likely that these few types are able to adequately represent a larger number of building types, this has not been verified.

The number of house types is a direct result of the scope of the experimental data used to develop the models for these guidelines (White and Ventura, 2007). Future research will allow for more building types to be incorporated.

Acknowledgments

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