Seismic Performance Assessment of Traditional Timber Himis Frames By Laboratory Testing and Capacity Spectrum Method

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SEISMIC PERFORMANCE ASSESSMENT OF TRADITIONAL TIMBER *HIMIS* FRAMES BY LABORATORY TESTING AND CAPACITY SPECTRUM METHOD

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ABSTRACT

The observations made by various researchers after many historic and contemporary earthquakes suggest that traditional timber-frame *hımış* houses, which still comprise a significant portion of the existing building stock in Turkey and in the Balkans, survived with insignificant damage in comparison to other construction types, hence they are seismically more resistant. However, these observations are based on qualitative evaluation and mostly lack a robust engineering approach. For this aim, a research project (no 106M499) funded by The Scientific and Technological Research Council of Turkey (TUBITAK) was carried out in order to assess and quantify the seismic performance of these buildings in relation to infill/cladding technique and frame geometry by means of full-scale frame tests and capacity/demand calculations.

The results showed that *hımış* frames will survive a design earthquake with any infill/cladding, although some are more advantageous than the others. In addition, the “window length” to “total length minus window length” ratio (WTLR) was shown to be a good indicator for rapid geometric evaluation.

INTRODUCTION

The general form and design principles of timber-frame *hımış* houses were first developed in Western Aegean Region (Kuban, 1995) and applied to a vast area, from the Southern Middle Anatolia to Black Sea Coasts of Romania, Crimea, Bulgaria, Macedonia, Bosnia Herzegovina, Mora, to Croatia and Hungary in the north, regardless of drastic differences in climate (Kuban, 1995; Eldem I, 1984) (Figure 1). *Hımış* houses have a hybrid construction system, where the ground floor is made of masonry with or without timber reinforcement, while the upper stories are comprised of timber frames.

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Figure 1. Examples to *hımış* houses in Turkey (first two photos, Safranbolu) and in Greece (last two photos, Athens and Chalkis, respectively)

The relevant literature reports many observations made after a number of historic and contemporary earthquakes, suggesting that the *hımış* structures behaved better under earthquake loading than masonry and reinforced concrete structures (e.g. Şahin-Güçhan, 2007; Langenbach, 2007; Gülhan and Özyörük Güney, 2000; Demirtaş *et al*., 2000). In those cases, where the timber buildings suffered structural damage, on the other hand, this was attributed mostly to a progressive damage that was caused by the collapse of masonry sections, such as chimney, ground floor or service walls (Şahin-Güçhan, 2007), to fires (Gürpınar *et al*.,1981), poor construction quality (Erdik et al., 2002) or liquefaction (Koçyiğit *et al*., 2002).

However, there are not many studies supporting these observations with more robust engineering means that will give further insight about the seismic behaviour of *hımış* structures. This paper reports the findings of a research project (no 106M499) funded by TUBITAK (The Scientific and Technological Research Council of Turkey) with the aim of assessing and quantifying the seismic performance of these buildings by means of full-scale frame tests and capacity/demand calculations.

FRAME TESTS

Firstly a number of typical layouts for test frames were selected from Safranbolu, a UNESCO World Heritage site since 1994 due to an important number of *hımış* houses listed as heritage buildings. This settlement is also located in 1st degree seismic zone. A total of 6 frames were chosen from the non-plastered facades that can be easily observed from outside and so as to cover the most common types in terms of geometrical configuration. 2 of these 6 frames were built using two different timber types, i.e. yellow pine and fir. Then, each of these 8 frames, built by local construction workers according to the traditional practices, was tested in their bare state under reverse-cyclic loading (Figure 2). After testing each frame, the damaged connections were repaired by using the same number and type of nails, and tested again with one of the two types of infill (brickwork, adobe masonry) or cladding technique (*bağdadi* and *şamdolma*) (Aktas *et al*., 2012).

There were certain randomness within the timber frames, tested under reverse cyclic lateral loading, that affect the behaviour. First of all, the number and driving angle of nails at each connection were not standard. The number of nails at each connection changed between 1 and 5, based on the construction worker's discretion. Also, the workmanship was not standard among the frame set, even though all frames were constructed, repaired and infilled/cladded by the same group of construction workers. The timber elements forming the frame were occasionally not connected well. Slightly out-of-plane connections were in some cases observed. The laths used for *bağdadi* and *şamdolma* covering were occasionally not nailed to each timber element of the frame they covered. These irregularities were not fixed before testing, since they are thought to be an intrinsic characteristic of traditional *hımış* frames.

Figure 2. Frame testing

The general information about the tested frames and the lateral displacement- lateral load graphs obtained at the end of each test are given in Table 1 and Table 2, respectively. One should notice that in some of the tests the ultimate strength level could not be reached due to laboratory safety regulations. To simulate the vertical load due to the roof, two load prisms were used, each of which 100 cm x 40 cm x 10 cm in size and weighing approximately 320 kg. These were held in suspension by means of a crane during tests for safety purposes. The tests were carried out in a load controlled way until the point where the lateral load-displacement curve leaned downward. After this point, the tests were continued in a displacement controlled way.

Table 1. Tested frames (all dimensions are in cm)

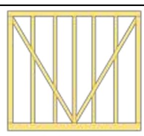
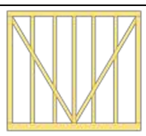
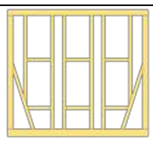
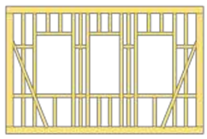
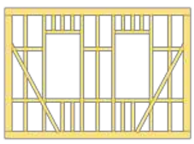
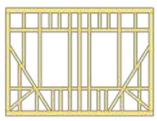
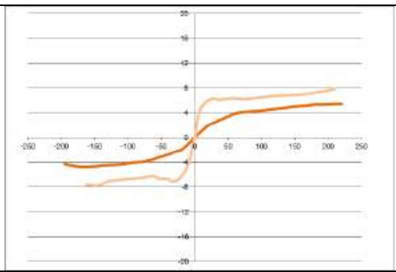
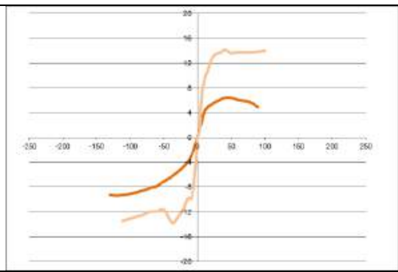
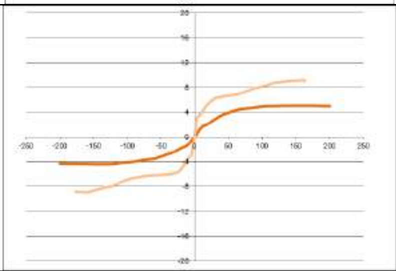
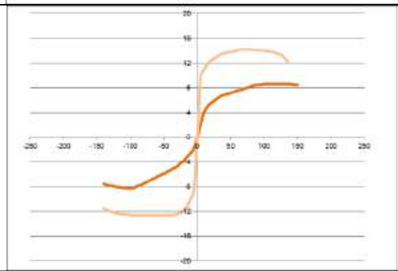
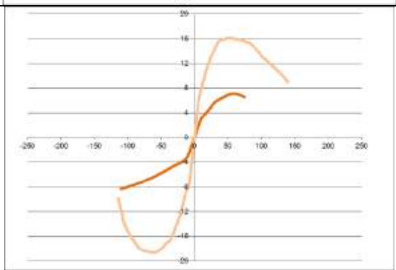
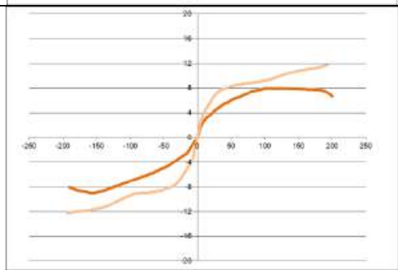
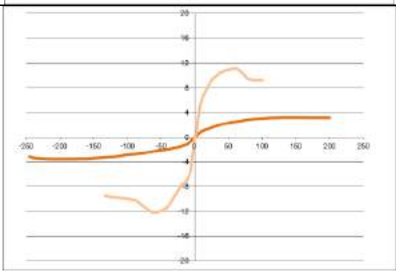
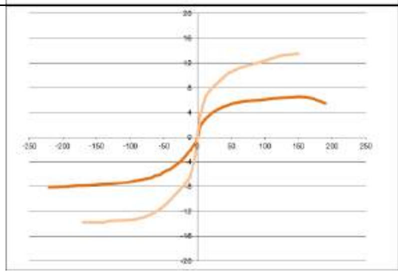
Frame		General information	Frame		General information
1		Yellow pine (H x L): 325 x 310 Windows: 135x67	2		Yellow pine (H x L): 360 x 330 No windows
3		Fir (H x L): 360 x 330 No windows	4		Fir (H x L): 325 x 310 Windows:135 x 67
5		Yellow pine (H x L): 330 x 370 Windows: 116 x 62	6		Yellow pine (H x L): 340 x 520 Windows: 157 x 93
7		Yellow pine (H x L): 340 x 485 Windows: 169 x 89.5	8		Yellow pine (H x L): 300 x 400 Windows: 156 x 75

Table 2. Tested frames (dark and light lines are for without- and with-infill/cladding states, respectively)

Infill/ Cladding	Frame	Load (kN)-displacement (mm)	Frame	Load (kN)-displacement (mm)
Adobe Masonry 	1		2	
Brick Masonry 	4		7	
Şamdolma 	3		6	
Bağdadi 	5		8	

In *bağdadi* cladding technique, 3-4 cm wide laths are nailed to the frame so as to leave a gap of several centimetres between each successive lath, while in *şamdolma* technique the laths are approximately 10 cm wide. After each infill and cladding process, the surface is plastered. All mortar, plaster and adobe blocks were prepared by local construction workers using traditional recipes. The 18th century solid frogged bricks were sourced from demolished historic buildings. In all tests, without or with infill/cladding, the governing damage mechanism took place at the nailed connections. At each loading cycle, nails at the opposite side of the loading direction, were partially pulled out, and at the next cycle they are driven back, until a point where the nails get pulled completely or get buckled leading to the loss of connection (Figure 3).

Figure 3. Failure at the nailed connections

The infill and cladding increased the lateral load strength of a timber frame, by the order of 1.74 and 2.25 times, respectively. However, the increase in the lateral load strength by infill and cladding is counterweighed by the increase in frame mass, in the order of 5.3 for frames with infill, and 3.4 for the frames with cladding. The only exception to this is the frame #5 that was cladded using *bağdadi* technique (Table 3).

Table 3. Average values for increase in lateral load strength, stiffness and weight of each frame with-infill/cladding

Frame	Infill / Cladding	Average Increase in Lateral Load Strength	Average Increase in Weight
1	Adobe Masonry	1.46	4.22
2		<1.83	6.79
4	Brick Masonry	2.06	4.57
7		<1.59	4.97
3	*Şamdolma*	>1.46	3.29
6		<2.27	4.18
5	*Bağdadi*	**<3.50**	**2.99**
8		1.84	3.09

The results that were obtained from these tests can be summarized as follows:

- The governing damage mechanisms take place always at the nailed connections. Therefore, timber type does not seem to be important because wood is not stressed to its strength limits;
- Infill/cladding increases the lateral load strength of a timber frame by on an average 2, however the increase in the lateral load strength is nearly always less than weight increase due to infill/cladding, and
- Among all the infill and cladding techniques, *bağdadi* seems to be the one that provides the best improvement in frame's behaviour, since it seems to satisfy the optimum combination of a high increase in lateral load strength and a low increase in weight (Aktas *et al*., 2012).

CAPACITY CALCULATIONS

Based on the results obtained from frame tests, one can think that infill/cladding will adversely affect the performance of a frame under seismic loading because a larger increase in weight than in lateral load strength will result in a higher seismic demand than the improved capacity. On the other hand, the seismic demand is also a function of structural period and damping ratio; therefore, a more detailed study for seismic demand to capacity ratio was needed. For this aim, the capacity spectrum method described in ATC-40 (1996) was used. ATC-40 has a well-established procedure for capacity based evaluation, which was originally designed for reinforced concrete structures. In the past, the capacity spectrum method (CSM) based evaluation of timber frame structures was discussed and made on analytical models (e.g. Kawai, 1999 and 2000; Hayashi *et al*., 2008).

The values assigned to each parameter for an ATC-40 based capacity spectrum method are as follows:

modal mass coefficient (α_1): 0.8

modal participation factor (PF_1): 1.4

structural behaviour type: Type C, which is defined as 'poor existing building' or "average existing building under long shaking duration"

seismic zone factor: 4, which is the worst case in a scale out of 4

soil profile type: E, which is the softest in a scale from A to E

earthquake hazard level: 1.25 as suggested for Zone 4 sites

near source factor: 1.0, assuming the closest distance to known seismic source is larger than 15 km distance.

According to the described procedure, first, the capacity curves were obtained by using the following equations:

$$S_a = \frac{V/_W}{\alpha_1} \quad (1)$$

$$S_d = \frac{\Delta_{roof}}{PF_1 \phi_{roof,1}} \quad (2)$$

where, S_a and S_d are spectral acceleration and spectral displacement, V is the base shear, W is building dead weight plus likely live loads, α_1 is the modal mass coefficient for the first natural mode, Δ_{roof} is top displacement, PF_1 is the modal participation factor for the first natural mode, and $\varphi_{roof,1}$ is amplitude of mode 1 at the roof level.

Some examples to the obtained capacity spectrums are given in Figure 4. A performance point could not be obtained for the frames #1, 4, 5, and 6 for their *without infill/cladding state*; therefore, they collapse under the maximum earthquake defined by ATC-40, while all frames appeared to be capable of surviving a design earthquake *with infill/cladding*. The results obtained in terms of the vibrational features of each frame without and with infill/cladding are shown in Table 3.

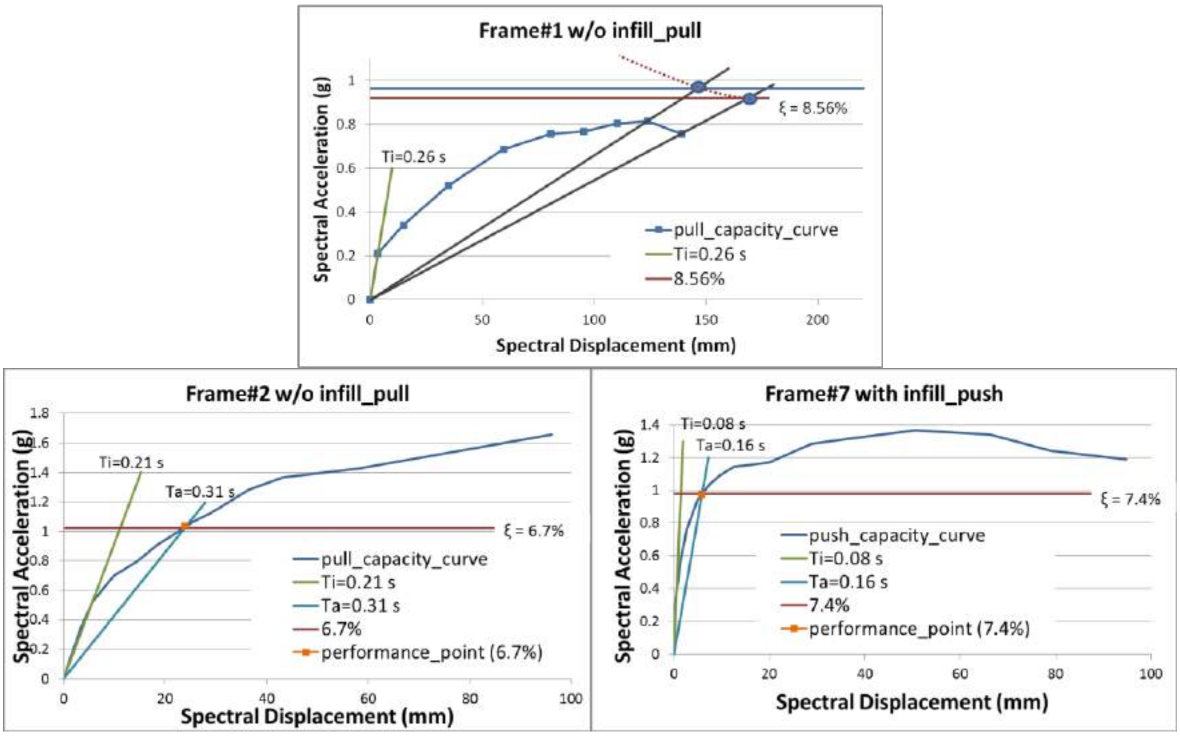


Figure 4. Examples to obtained capacity spectrums without (upper row) and with (lower row) performance points

Table 4. Results of the capacity calculations for each frame without and with infill/cladding

Frame	Without-Infill State		With-Infill/Cladding State	
	push	pull	push	pull
1	Ti=0.27 s	Ti=0.26 s	Ti=0.18 s Ta= 0.78 s Sd= 135 mm ξ= 8.8%	Ti=0.22 s Ta= 0.71 s Sd= 113 mm ξ= 9.2%
2	Ti=0.17 s Ta=0.26 s Sd= 17 mm ξ= 7.2%	Ti=0.21 s Ta=0.31 s Sd= 24 mm ξ= 6.7%	Ti=0.12 s Ta= 0.17 s Sd= 7.5 mm ξ= 7.1%	Ti=0.14 s Ta=0.16 s Sd= 6.5 mm ξ= 6.3%
3	Ti=0.18 s Ta=0.31 s Sd= 25.5 mm ξ= 6.4%	Ti=0.20 s Ta=0.39 s Sd= 36.9 mm ξ= 7.1%	Ti=0.10 s Ta=0.13 s Sd= 4 mm ξ= 7.3%	Ti=0.11 s Ta=0.16 s Sd= 6.6 mm ξ= 8%
4	Ti=0.29 s	Ti=0.40 s	Ti=0.17 s Ta=0.53 s Sd= 65 mm ξ= 8%	Ti=0.22 s Ta=0.62 s Sd= 93.2 mm ξ= 7.6%
5	Ti=0.42 s	Ti=0.43 s	Ti=0.16 s Ta=0.23 s Sd= 13.5 mm ξ= 6.9%	Ti=0.16 s Ta=0.23 s Sd= 12.9 mm ξ= 7.3%
6	Ti=0.33 s	Ti=0.18 s	Ti=0.21 s Ta=0.56 s Sd= 72.5 mm ξ= 8.2%	Ti=0.21 s Ta=0.54 s Sd= 68 mm ξ= 8.1%
7	Ti=0.22 s Ta=0.32 s Sd= 26 mm ξ= 7.0%	Ti=0.26 s Ta=0.42 s Sd= 46 mm ξ= 6.6%	Ti=0.08 s Ta=0.16 s Sd= 5.6 mm ξ= 7.4%	Ti=0.11 s Ta=0.19 s Sd= 8.9 mm ξ= 7.5%
8	Ti=0.20 s Ta=0.45 s Sd= 49 mm ξ= 7.7%	Ti=0.25 s Ta=0.40 s Sd= 41 mm ξ= 6.6%	Ti=0.11 s Ta=0.18 s Sd= 6.3 mm ξ= 6.8%	Ti=0.09 s Ta=0.16 s Sd= 6.5 mm ξ= 7.3%

The frames#1, 4, 5, and 6, for which a performance point could not be obtained, have larger window openings. Comparing the Sd values obtained for frames with infill or cladding also revealed interesting results; the frames with a "window length" to "total length minus window length" ratio (WTLR) larger than 2/3, which did not yield a performance point, exhibited also much higher Sd values than the other frames. The same pattern can be clearly observed also for ductility factors (Figure 5 and Figure 6).

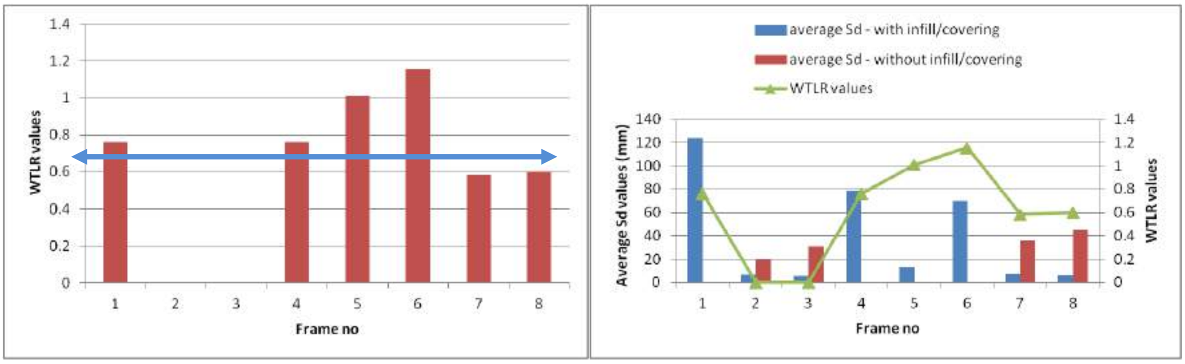


Figure 5. Comparison of frames in terms of spectral displacement and WTLR values

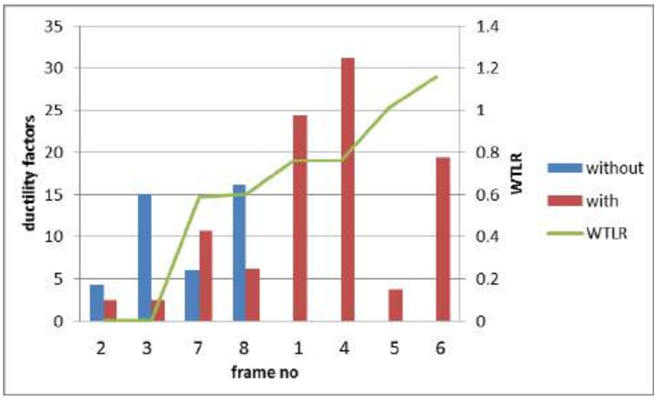


Figure 6. Ductility factors for each frame without and with infill/cladding and WTLR values

In order to calculate a capacity/demand ratio for each frame, the seismic demand on each frame was also determined. The demand calculations were carried out based on Eurocode 8 (2004), using the equation below:

$$F_b = S_d(T_1) * m * \lambda \quad (3)$$

where, F_b stands for base shear force, i.e. seismic demand, m for the total mass of the building and λ for the correction factor, which was taken equal to 1.0 as suggested by Eurocode 8 (2004) and finally $S_d(T_1)$ for the ordinate of the design spectrum at period T_1. The results obtained for each frame are summarized in Table 5.

Table 5. Capacity and demand values calculated for each frame without and with infill/cladding in the linear and non-linear states (NA, L, NL, and *şd* stand for Not Applicable (as no performance point was obtained), Linear, Non-Linear and *şamdolma* respectively)

frame	WTLR		WITHOUT INFILL/CLADDING						Infill/ cladding	WITH INFILL/CLADDING					
			load bearing capacity (kN)		demand Fb (kN)		capacity/ demand			load bearing capacity (kN)		demand Fb (kN)		capacity/ demand	
			L	NL	L	NL	L	NL		L	NL	L	NL	L	NL
2	0	push	3.03	6.38	10.57	5.56	0.29	1.15	adobe	6.93	14.18	16.97	9.04	0.41	1.57
	0	pull	2.92	>9.36	10.57	5.56	0.28	>1.68		7.24	13.80	16.97	9.04	0.43	1.53
3	0	push	0.98	7.07	10.92	5.75	0.09	1.23	*şd*	5.90	16.03	13.91	7.10	0.42	2.26
	0	pull	1.97	>8.33	10.92	5.75	0.18	>1.45		6.89	18.66	13.91	7.41	0.50	2.52
7	0.58	push	2.96	8.57	12.17	6.40	0.24	1.34	brick	3.92	14.18	19.88	10.59	0.20	1.34
	0.58	pull	2.10	8.13	12.17	6.40	0.17	1.27		4.43	12.50	19.88	10.59	0.22	1.18
8	0.60	push	2.00	6.38	11.27	5.93	0.18	1.08	*bağdadi*	3.99	>13.43	12.66	6.74	0.31	>1.99
	0.60	pull	1.01	>8.47	11.27	5.93	0.09	>1.43		3.13	>13.33	12.66	6.74	0.25	>1.98
1	0.76	push	1.05	>5.42	10.96	NA	0.10	NA	adobe	3.92	>7.82	7.52	7.52	0.52	>1.04
	0.76	pull	1.21	4.7	10.96	NA	0.11	NA		3.95	>8.81	7.52	7.52	0.53	>1.17
4	0.76	push	1.05	>5.01	10.96	NA	0.10	NA	brick	2.00	>9.19	7.87	7.87	0.25	>1.17
	0.76	pull	1.01	>4.30	10.96	NA	0.09	NA		2.79	>8.95	7.87	7.87	0.35	>1.14
5	1.01	push	2.00	3.17	11.55	NA	0.17	NA	*bağdadi*	4.95	11.10	6.84	6.84	0.72	1.62
	1.01	pull	1.97	>3.47	11.55	NA	0.17	NA		5.18	12.23	6.84	6.84	0.76	1.79
6	1.16	push	2.00	7.96	13.61	NA	0.15	NA	*şd*	3.03	11.79	9.99	9.99	0.30	1.18
	1.16	pull	1.15	8.88	13.61	NA	0.08	NA		3.40	12.23	9.99	9.99	0.34	1.22

The results that can be drawn at the end of capacity calculation can be summarized as follows:

- All timber frames with infill/cladding yielded a performance point regardless of the infill material or cladding technique, which means they will survive a design earthquake. However, the capacity to demand ratios calculated for each frame in the linear range is below 1. Therefore, frames, without or with infill/cladding, are not capable to bear seismic demand in the linear range and all pass to the non-linear range with a certain amount of damage.

- The “window length” to “total length minus window length” ratio (WTLR) was shown to be a good indicator for rapid geometric evaluation. The frames with no infill/cladding and with WTLR ratio smaller than 2/3 have resulted a performance point while others collapsed under maximum earthquake loading according to ATC-40. The frames with infill/cladding and with WTLR ratio larger than 2/3 resulted in larger spectral displacement demands.
- *Bağdadi* type cladding has shown to be superior to other infill (brick, adobe) and cladding (*şamdolma*) types. Furthermore, *bağdadi* type cladding was shown to alter the bad performance of bare frames with WTLR value larger than 2/3.
- For the frames which a performance point was obtained for their bare states, the factors of safety was approximately doubled with cladding, while infill increase the capacity to demand ratio only by an average order of 1.3.
- The average ductility factor value for the frames without infill or cladding is 10.4, while this value is 17.2 and 8.0 for frames with infill and cladding, respectively. Also frames with high WTLR values have high ductility factors.

CONCLUSIONS

Results showed that *hımış* houses can survive a design earthquake with a certain amount of damage (without complete collapse) *provided that the masonry ground floor (and other masonry sections of the building, if any) is strong enough to bear seismic loading and the timber skeleton is well connected to the masonry ground floor*. Another important issue that should be considered is that the frames that were tested within content of this study may exhibit some scattered features in terms of number of nails, driving angles of nails and workmanship, which have a direct effect on the structural behaviour under seismic loading. Also, the tests and analyses reported here do not take the material degradation of exiting *hımış* buildings into consideration. The conclusions drawn here relate more to a certain building technology, rather than to the existing building stock. On-site tests and further analyses are needed to evaluate the seismic vulnerability of *hımış* buildings in district level.

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REFERENCES

Kuban, D. (1995). The Turkish Hayat House. Istanbul: T.C. Ziraat Bankası Kültür Yayınları.

Eldem, S. H. (1984). Türk evi : Osmanlı dönemi / Turkish houses : Ottoman period. İstanbul: Türkiye Anıt, Çevre, Turizm Değerlerini Koruma Vakfı

Şahin Güçhan N (2007) “Observations on Earthquake Resistance of Traditional Timber Framed Houses in Turkey,” *Building and Environment*; 42: 840-851

Langenbach R (2007) “From “Opus Craticium” to the “Chicago Frame”: Earthquake – Resistant Traditional Construction,” *International Journal of Architectural Heritage*, 1: 29-59.

Gülhan, D., & Özyörük Güney, İ. (2000). “The Behaviour of Traditional Building Systems Against Earthquake and Its Comparison to Reinforced Concrete Frame Systems: Experiences of Marmara Earthquake Damage Assessment Studies in Kocaeli and Sakarya”. *Earthquake-Safe:Lessons to be Learned from Traditional Buildings*. Istanbul: ICOMOS.

Demirtaş, R., Iravul, Y., Erkmen, C., Baran, B., Yaman, M., Baykal, M., et al. (2000). “ÇankırıDepremi - 06 Haziran 2000 Orta (Çankırı) Depremi”. *Jeoloji Mühendisleri Odası Haber Bülteni* , 6-15.

Gürpınar, A., Erdik, M., & Ergünay, O. (May 24-28, 1981). “Sitting and Structural Aspects of Adobe Buildings in Seismic Areas”. *International Workshop on Earthen Buildings in Seismic Areas*, (pp. 139-158). Albuquerque, New Mexico.

Erdik, M., Sesetyan, K., Demircioglu, M. B., Celep, U., Biro, Y., & Uckan, E. (2002b). "Special Earthquake Report from the May 2002 Newsletter: Preliminary Observations on the Sultandağı, Turkey, Earthquake of February 3, 2002". Retrieved March 14, 2014, from Earthquake Engineering Research Institute, Learning From Earthquakes - Turkey: http://www.eeri.org/lfe/pdf/Turkey_Sultandagi_Insert_May02.pdf

Koçyiğit, A., Bozkurt, E., Kaymakcı, N., & Şaroğlu, F. (2002, February 18). "3 Şubat 2002 Çay (Afyon) Depreminin Kaynağı ve Ağır Hasarın Nedenleri: Akşehir Fay Zonu (Jeolojik Ön Rapor)". Retrieved March 12, 2014, from http://www.metu.edu.tr/~akoc/Afyon.pdf

Aktaş YD, Akyüz U, Türer A, Erdil B, and Şahin Güçhan N (2012) "Seismic Resistance Evaluation of Traditional Ottoman Timber-Frame Hımış Houses: Frame Loadings and Material Tests," *Engineering Spectra* (in press)

Applied Technology Council (ATC) (1996) Seismic Evaluation and Retrofit of Concrete Buildings, Report No. ATC-40, Redwood City: CA.

Kawai N. (1999) "Prediction methods for earthquake response of shear walls", *Pacific Timber Engineering Conference,* Rotorua, New Zealand, pp. 317–324.

Kawai N. (2000) "Application of Capacity Spectrum Method to Timber Houses Considering Shear Deformation of Horizontal Frames", *World Conference on Timber Engineering*, *Whistler, BC, Canada*.

Hayashi Y., Nii A., Morii T. (2008) "Evaluation of Building Damage Based on Equivalent-Performance Response Spectra", *14th World Conference on Earthquake Engineering,* Beijing, China

Eurocode 8 (2004) Design of Structures for Earthquake Resistance Part 1: General Rules, Seismic Actions and Rules for Buildings. Brussels: CEN: European Committee for Standardization.