Technical Note:

Pullback Test for Seismic Performance Evaluation of Safe Rooms

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Abstract: *Pullback test has no scrupulous theoretical establishment. It is based on the hypothesis that the response of the structure can be related to the response of an equivalent single degree-of-freedom (SDOF) system. This implies that the response is controlled by a single mode. In fact, the steel frame of each safe room, which is introduced within the unreinforced masonry buildings for protecting the lives of residents in catastrophic earthquake failures, contains a SDOF structural system. In pullback test, the steel frame carries its gravity load first, and then it will be pushed under an incremental lateral roof displacement pattern, which is imposed to its center of mass. This paper expresses the results of 13 pullback tests executed by the authors on the steel frames of safe rooms. The results show that pullback test is a practical method for seismic performance evaluation of safe rooms. Also the performance of these frames located in a collapsing three storey masonry building is presented with favorable conclusions. In fact, the results of pullback test of the safe room located at the ground-floor level were compared with the requirements of Iranian code for seismic resistant design and it was concluded that the steel frame had an acceptable performance against seismic effects.*

Keywords: *earthquake, masonry building, safe room, steel frame, vibration, pullback test.*

1. Introduction

Catastrophic earthquakes appear in the headlines with discomforting frequency, causing thousands of lives to be lost especially in masonry buildings. This truly global phenomenon has begun to be understood, and considerable emphasis is being placed on the analytical studies supported by experimental studies both in the laboratory and in the field in an effort to prevent much of this loss of life. It is worth noting that brick masonry has been used as a load bearing material for centuries. In gravity structures constructed by this material, the level of gravity stresses are low and the factor of safety against compression failure is high [1]. Moreover, there is no need for high technology to construct masonry buildings; as a result, they are not expensive. Because of these advantages, the masonry buildings constitute a large portion of the building stock in the world. But masonry structural elements can not resist earthquake effects because their bond tensile strength is too low [2-7]. This is the common problem and the wide-spread concern of structural engineers and building officials, as well as owners, lenders and insurers especially in areas of high seismicity.

Safe room is the name of a new method, which is regarded as economical and practical, and it can be used for lowering earthquake life losses in masonry buildings. In this method, some safe areas having a good balance of safety versus cost will be prepared inside the building and the existing load carrying system of the structure will not change. The practical method for making these safe areas inside a building is to introduce some prefabricated moment resisting steel frames there. This paper studies the load bearing and drift capacities of the steel frames based on pullback tests. These one-story, one-bay steel frames are characterized by rigid connections between the beams and columns that force the entire frame to deform when subjected to lateral loads. Baker et al. have introduced a similar shelter for saving the human lives against bomb explosions [8]. This shelter, which is like a table, is capable of accommodating a family of two adults and two children in such a way that if the house collapses completely, due to a near miss

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from a large bomb, the occupants will not be crushed by the derbies and they will be able to escape or be rescued in a short time.

It is worth noting that explosion or blasting often occurs with a warning. Therefore, safe rooms or defence shelters are very common for explosion protection. However, earthquake strong shakings usually occur without any warnings and they last for a few seconds and the occupants who are slipping will have no time to get into the safe rooms once strong shakings start. In other words, the first and best place for installing the steel frames is inside the bedrooms.

In an ideal world there would be no debate about the appropriate method of demand prediction and performance evaluation of the steel frames of safe rooms at low performance levels. Clearly, inelastic time history analysis that predicts with adequate reliability the forces and cumulative deformation demands in every element of the structural system is the ultimate resolution. The accomplishment of this solution requires the availability of a set of earthquake records that account for the reservations and differences in frequency characteristics, severity, and duration due to rupture characteristics and distances of the various faults that may cause motions at the place [9].

It should be worked towards this ultimate resolution, but it is also required to recognize the restrictions of today's states of knowledge and practice. Recognizing these restrictions, the task is to perform an evaluative process that is relatively simple but captures the crucial features that significantly affect the performance purpose. In this context, the accuracy of demand prediction is attractive, but it may not be necessary, since accurate seismic inputs are not known. Using one pullback test for each group of the safe rooms, which is the subject of this paper, serves this purpose. In pullback test, the steel frames carry their gravity loads first, and then they will be pushed under incremental lateral roof displacement patterns, which are imposed to their centers of masses. According to the results of these tests, base shear versus lateral roof displacement graphs will be plotted. The initial slops of the graphs, which are stiffness coefficients, the ultimate carried base shears, and the maximum lateral roof displacements are utilized in this research to evaluate the seismic performance of safe rooms.

2. Pullback test

The theoretical assumption of pullback test is based on the hypothesis that the response of the system can be related to the response of an equivalent single degree-of-freedom (SDOF) structure. This implies that the response is controlled by a single mode, and that the shape of this mode does not change throughout the time history response. In fact, each safe room contains a SDOF structural system; therefore, this test can be used for its seismic performance evaluation. In other words, both assumptions above are approved in the simple structures of safe rooms. Although those hypotheses are incorrect in multi degree-of-freedom (MDOF) structures, previous investigations have indicated that these assumptions lead to fine predictions of the maximum seismic response of MDOF structures if their response is conquered by a single mode [10-12].

3. Implementation of pullback test

The process is to represent the response of the three-dimensional full size structures that accounts for all important response characteristics. Gravity loads followed by incremental lateral roof displacements should be applied on the structures in patterns that represent around the relative inertia forces generated at roof levels. The structures have to be pushed under these lateral roof displacement patterns to maximum drifts that are associated with the collapse of the systems. These final deformations are used as estimates of the deformation capacities, which need to be compared to available inelastic drifts.

The details of the gravity tests and the final constructional drawings were published earlier [13,14]. It is worth noting that 13 full-scale tests, which were not explained in previous publications, were pullback tests. These tests were designed to realize whether the collapses of the safe rooms were caused by brittle failure modes in elements and connections that were important parts of the gravity and lateral load paths or not. Thus, the emphasis in the experimental work was on: confirmation that sufficient load paths exist; confirmation that the load paths stay sound at the deformation associated with the maximum displacement levels; confirmation that rigid connections stay capable of transferring loads between beams and columns; confirmation that individual elements that may fail in brittle modes are not overloaded; confirmation that local failures do not pose the total collapse of the system.

In final pullback test (Test 29), the lateral load was applied 11 days after imposing the gravity loads and the maximum temperature variation during these 11 days was 20 °C. This time duration is assumed to be enough for rescuing the people who may be trapped in safe rooms at the time of an earthquake. In the other 12 pullback tests, the lateral loads were applied on the structures immediately after imposing the gravity loads. As a result of the delay in applying the lateral load of the final test, the stiffness coefficient obtained from this test was lower than those of the other tests. Base shear versus lateral roof displacement graph of the final test can be seen in Fig. 1.

The assessed parameters for evaluating the seismic performance of the steel frames were stiffness coefficient, k, and natural period of vibration, T. Natural period of the structure can be calculated by the following equation:

$$T = 2\pi\sqrt{\frac{m}{k}} \tag{1}$$

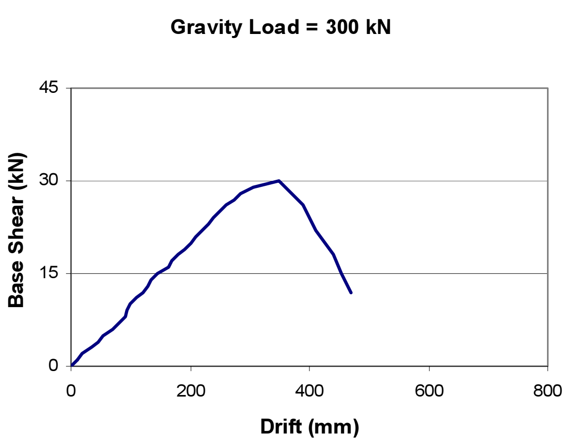

Fig. 1. Experimental results of the final pullback test

where m is vibrating mass. The T value of each vibrating mass is obtained by substituting the experimental k value in Eq. (1). It is worth emphasizing that according to standard No. 2800-05 [15], there are no limitations for the amount of analytical T values above, which are used for estimating the lateral drifts of the structures.

4. Seismic performance evaluation

In this part of the research, the lateral force-horizontal displacement relationships of the safe rooms were obtained to assess their seismic performance. For this purpose, the pullback tests were conducted and the incremental lateral displacements were imposed to the steel frames and the applied forces in each step were measured. The results of these tests including stiffness coefficients, natural periods of vibration, ultimate base shears, and maximum lateral drifts can be seen in Table 1. As published earlier [14], the 100, 200, and 300 kN gravity load structures of this table were the steel frames, which carried the gravity loads of one, two, and three floors respectively.

Standard No. 2800-05 [15] gives the following equation for measuring the lateral seismic load applied to the structure.

$$F = \frac{A.B.I}{R}.W \tag{2}$$

where A is design base acceleration, B is response coefficient, I is importance coefficient, R is performance coefficient, and W is seismic weight of the building. In this standard B is related to the natural period of vibration of the building, kind of the ground, and the seismic zones. Table 2 shows the calculated lateral loads of the steel frames in an area of high seismicity and the ground type 2. It is clear that all the seismic loads, F, of this table are lower than ultimate lateral shear forces carried by the safe rooms, which can be seen in Table 1.

The elastic SDOF displacement demand can be computed as $D_e=F/k$. The results of this computation can be seen in Table 3. These elastic displacement demands are the base lines for predicting the inelastic displacement demands,

Table 1. Experimental results of pullback tests

Test	W (N)	k (N/mm)	T (sec)	Ultimate base shear (N)	Maximum lateral drift (mm)
Test 14	300,000	86.4	3.7	19,000	320
Test 15	300,000	111.8	3.25	32,000	430
Test 16	300,000	103.2	3.39	24,000	470
Test 17	300,000	110.5	3.27	38,500	540
Test 18	200,000	100	2.81	24,000	620
Test 19	200,000	113.6	2.64	41,000	850
Test 21	200,000	94.4	2.89	21,000	420
Test 23	200,000	100	2.81	17,000	370
Test 25	200,000	100	2.81	30,000	420
Test 20	100,000	92.3	2.07	34,000	820
Test 22	100,000	114.3	1.86	23,000	570
Test 24	265,000	114.3	3.02	24,000	290
Test 29 (final)	300,000	100	3.44	30,000	470

Table 2. Seismic loads applied to the steel frames

Test	T (sec)	B Value	C=A.B.I/R*	W (N)	F=C.W (N)
Test 14	3.7	0.65	0.046	300,000	13,800
Test 15	3.25	0.71	0.050	300,000	15,000
Test 16	3.39	0.69	0.048	300,000	14,400
Test 17	3.27	0.71	0.050	300,000	15,000
Test 18	2.81	0.79	0.055	200,000	11,000
Test 19	2.64	0.82	0.057	200,000	11,400
Test 21	2.89	0.77	0.054	200,000	10,800
Test 23	2.81	0.79	0.055	200,000	11,000
Test 25	2.81	0.79	0.055	200,000	11,000
Test 20	2.07	0.97	0.068	100,000	6,800
Test 22	1.86	1.04	0.073	100,000	7,300
Test 24	3.02	0.75	0.053	265,000	14,045
Test 29 (final)	3.44	0.69	0.048	300,000	14,400

*A=0.35; I=1; R=5

which need to be accomplished with due consideration given to the yield strength and hysteretic characteristics of the SDOF system. Both effects of yield strength and hysteretic

Table 3. Elastic and inelastic lateral drifts of the safe rooms

Test	k (N/mm)	F (N)	D_e= F/k (mm)	D_{ine}=0.7R.D_e (mm)	Accepted
Test 14	86.4	13,800	159.7	559	No
Test 15	111.8	15,000	134.2	469.7	No
Test 16	103.2	14,400	139.5	488.3	No
Test 17	110.5	15,000	135.7	475	Yes
Test 18	100	11,000	110	385	Yes
Test 19	113.6	11,400	100.4	351.4	Yes
Test 21	94.4	10,800	114.4	400.4	Yes
Test 23	100	11,000	110	385	No
Test 25	100	11,000	110	385	Yes
Test 20	92.3	6,800	73.7	258	Yes
Test 22	114.3	7,300	63.9	223.7	Yes
Test 24	114.3	14,045	122.9	430.2	No
Test 29 (final)	100	14,400	144	504	No

characteristics can be accounted through adaptation factors applied to the elastic displacement demands. It is worth noting that much information has been reported on the effect of yield strength on SDOF seismic demands [16-23]. In this research, the equation suggested by standard No. 2800-05 [15] is utilized for calculating the inelastic displacement demands of the investigated safe rooms according to the elastic results. This equation is:

$$D_{ine}=0.7R.D_e \quad (3)$$

where R is performance coefficient, D_e is elastic displacement demand and D_{ine} is inelastic displacement demand. According to this equation, once the R-factor is known, the SDOF inelastic displacement demand can be computed. Standard No. 2800-05 [15] suggests that the R-factor of ordinary moment resisting steel frames is equal to 5. Table 3 gives the inelastic displacement demands of the safe rooms too. Regarding inelastic drift, the results of Tables 2 and 3 reveal that, only the structure of Test 17 is acceptable for carrying the 300 kN gravity loads. Also, all the tests except Test 23 were acceptable for 200 kN gravity load structures. The steel frames carrying 100 kN gravity loads had no problem considering inelastic drift capacities. The final pullback test (Test 29) shows applying the delayed lateral load affects the result and the structure can not carry the inelastic lateral drift. In other words, the occupants who may be trapped in these frames should be rescued as soon as possible.

5. Increasing the Lateral drift capacity

In earthquake-resistant design, considering the strength of a member is not sufficient, and it must also have a preserve of ductility. In fact, ductile materials are extremely enviable for earthquake-resistant design because earthquake design should satisfy the following basic objectives: to prevent loss of life and serious injury and to prevent buildings from collapse and dangerous damage under a maximum seismic load; to ensure structures against severe damages under moderate to heavy ground motions. Therefore, the strength alone cannot create an earthquake-resistant design. Earthquake resistance requires energy absorption, which means the structure should have conventional ductility as well as strength. The ductility of the structure can be

Fig. 2. Utilizing dowel plates in beam to column moment connections

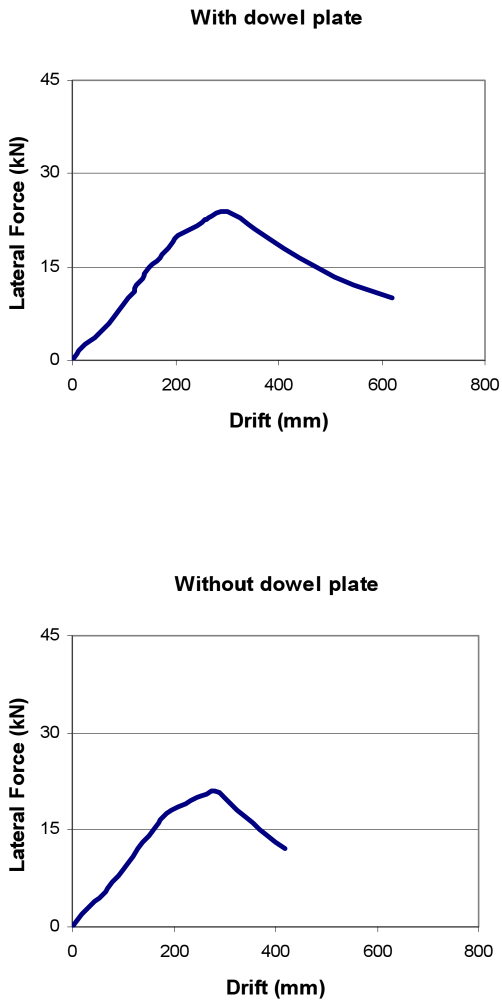

Fig.3. Effect of dowel plates on the drift capacity of the investigated safe room

visualized as its capacity to experience large deformations without noticeably losing its load-carrying capacity. If the designed structure has such ductility, it will not collapse even if it is seriously damaged. Therefore, in addition to seismic strength design, the ductility of the structure should be considered.

The required ductility can be achieved by appropriate framing and connection details. Because of the increased emphasis placed on ductility, the researchers decided to check the influence of adding some dowel plates in the beam to column connections (Fig. 2). Fig. 3 shows the effect of these plates on the lateral drift capacity of the steel frame. The lateral load versus lateral displacement graph reveals that the roof drift ratio (lateral displacement at the top divided by the total height) and energy absorption capacity of the system improved about 50% after choosing this connection detail. Therefore, as shown in construction drawings [14], the dowel plates should be utilized in all the steel frames of safe rooms.

6. Case study

A three storey demolishing masonry building helped determine the true performance of safe rooms. The constructional details of the building and the installed steel frames were published earlier [14]. After manufacturing the three storey masonry building, the steel frames of safe rooms were installed in the southern rooms of all the floors. It should be noted that all the installed frames were concentric.

Jack arch masonry slabs were utilized for constructing the entire three floors of the building. This flooring system was developed in Britain more than 100 years ago, and it was used widely to cover large floor areas in different kinds of buildings. This technique extended eastwards and gradually became a popular flooring system in parts of East Europe and the Middle East. Due to its low expenditure, constructional speed, and technical simplicity, jack arch slabs are still very popular in some parts of Iran [3].

The performance of the jack arch slab in a number of recent earthquakes in the Middle East, particularly in Iran, has generally been poor. In this research, to overcome this shortcoming, some horizontal bracings were used for

Fig. 4. Pullback test of the safe room located at the ground-floor level

connecting the bottom flanges of the beams. Utilizing these bracings is a practical method for improving the in-plane rigidity of the existing masonry slabs.

In this part of the research, the lateral force-horizontal displacement relationship of the safe room located at the ground-floor level was obtained to assess its seismic performance. For this purpose, a pullback test was conducted, and the incremental lateral displacements were imposed to the steel frame, and the applied force in each step was measured. Fig. 4 shows the safe rooms a few moments before destruction. In Fig. 5, the total base shear in each step is plotted against the accompanied roof level lateral drift.

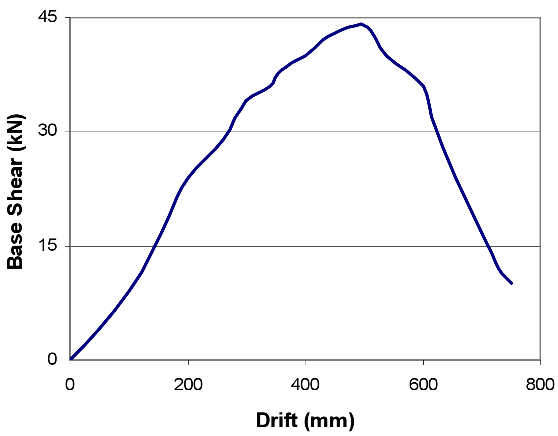

Fig. 5. Test result of the safe room located at the ground-floor level

a) Displacement measurement

b) Force measurement

Fig. 6. Instruments used for measuring the lateral displacements and forces

According to this figure, the stiffness coefficient (k) was 120 N/mm. The natural period T calculated according to the k value above was 2.89 sec. Fig. 6 shows the instruments used for measuring the lateral displacements and forces imposed to the cable, which is connected to the center of mass of the system.

According to Eq. (2) extracted from Standard No. 2800-05 [15], with reference to the experimental T value, the calculated lateral load F was 11880 N. Fig. 5, which is obtained from pullback test, shows that the ultimate lateral load capacity of the system was 44000 N. It means, the lateral load capacity of the system was more that three times of the analytical seismic force predicted by standard No. 2800-05 [15]. This result pronounces the adequacy of strength capacity of the steel structure to accommodate the seismic loads.

The elastic SDOF displacement demand of the safe room located at the ground-floor level can be calculated as $D_e=F/k$, and the result is *99* mm. This elastic displacement demand is the base line for predicting the inelastic displacement demand. According to Eq. (3), the inelastic displacement demand of the investigated safe room becomes 347 mm. Fig. 5 shows the maximum lateral drift of the system was 750 mm. In other words, the inelastic displacement demand was less than half of the maximum lateral drift capacity. This result pronounces the adequacy of displacement capacity of the steel structure to accommodate the distortions generated by seismic loads and aftershocks.

7. Conclusions

Pullback test is a practical method for evaluating the seismic performance of safe rooms because each safe room contains a SDOF structure, and applying the incremental lateral drift on its center of mass is quite straightforward. To estimate the natural period of vibration of a safe room as exact as possible, the stiffness obtained from pullback test should be used. The pullback tests of the investigated structural systems of the safe rooms located at different floors, which carry different gravity loads, show the ductility, lateral stability and strength capacity of the structural systems quite satisfactorily. Therefore, pullback test can be properly used to verify the capability of safe rooms for accommodating the distortions generated by seismic loads and aftershocks.

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