Structural Engineering and Mechanics, Vol. 46, No. 3 (2013) 387-402

DOI: http://dx.doi.org/10.12989/sem.2013.46.3.387

Fatigue experiments on steel cold-formed panels under a dynamic load protocol

Antonio J. García-Palencia[1a] and Luis A. Godoy[*2]

[1]*Department of Civil Engineering, University of New Hampshire, Durham, NH 03824, USA*
[2]*Structures Department, FCEFyN, National University of Córdoba, and CONICET,*
P.O. Box 916, Cordoba 5000, Argentina

(Received October 25, 2012, Revised February 28, 2013, Accepted April 9, 2013)

Abstract. A dynamic load protocol has been used to experimentally simulate fatigue behavior in cold-formed metal panels with screwed connections under wind loading. The specific protocol adopted is an adaptation of SIDGERS, originally developed for non-metallic membranes, which is composed of levels each under increasing load values. A total of 19 tests were performed on 3.35 m long by 0.91 m wide panels, identified as Type B-wide rib and Type E, both with screw connections at the edge and at the center, thus conforming two-span specimens. In some configurations the panels were fixed at the valleys, whereas crest-fixed connections were also investigated. Reinforcing the connections by means of washers was also investigated to evaluate their efficiency in improving fatigue capacity. The experimental results show maximum load capacities in improved connections with washers of approximately twice of those with classical connections.

Keywords: old-formed plates; connections; dynamic load protocol; fatigue failure; roof systems; SIDGERS; steel; testing

1. Introduction

Light weight systems, which are built using thin-walled metal components as roof and wall cladding, are widely used in the construction of commercial and industrial buildings and storehouses due to their simple installation and low cost (Newman 2003). In typical configurations the cladding is formed by thin-walled metal plates with several possible cross-sections, which are connected to beam and frame systems by means of screws.

However, the claddings of such buildings turn out to be highly vulnerable under wind loads due to hurricanes; and to a large extent, the vulnerability of such systems depends on their connections (López and Godoy 2005). In the event of a hurricane there are several parties interested in understanding the risk in terms of both the natural hazard and the vulnerability of the construction, including the owner of the construction, insurance companies, rating laboratories, and researchers.

Vulnerability of the construction depends on both structural characteristics and construction practices. Baskaran *et al*. (2006) found that air leakage of the structural deck strongly influences

∗Corresponding author, Professor, E-mail: lgodoy@efn.uncor.edu
[a]Ph.D. Candidate, E-mail: ajw77@wildcats.unh.edu

the performance under wind uplift. Although there are some tools for estimating vulnerability of constructed facilities, there is a need to better understand the structural behavior of such structures and their components under repeated loads produced by wind gusts due to hurricanes.

As of today, several standards have been adopted by the insurance industry to evaluate the load capacity of roof and cladding systems in industrial buildings in the United States, such as FM 4470 (Factory Mutual Research 1988) and UL 580 (Underwriters Laboratories 1996). Other standards used worldwide include the Australian TR440 (Experimental Building Station 1978), European UEAtc 551 (Gerdhardt and Kramer 1988), and NBI 192-90 (Norwegian Building Research Institute 1990). More recently, Henderson *et al*. (2009) and Kopp *et al*. (2010) presented a testing method for full-scale structures focused to residential and other light frame systems; this method simulates the spatial and temporal variations due to wind-induced turbulences on the buildings surfaces. The authors proposed the use of "pressure loading actuators" and airbags, in which the set-up is similar to the system presented by Cook *et al*. (1998) at the British Research Establishment. Also, Baskaran *et al*. (2012) evaluated the wind uplift performance of noncomposite and composite metal roof assemblies under the protocol given by the Canadian Standards Association CSA A123.21-04. This protocol in particular, provides a better representation of wind effects by inducing fatigue on roofing components and also varying wind gust intensity and is based on the work done by the Special Interest Group for Dynamic Evaluation of Roofing Systems (SIDGERS) (Baskaran *et al*. 1999).

Morgan and Beck (1977), Beck and Stevens (1979) were among the first researchers to report studies of wind-induced fatigue damage in metal roof cladding fixed with screws; they conducted fatigue tests to simulate the effect of hurricane winds, and evaluated the capacity of the roof plates connected with self-drilling and self-tapping screws. It was concluded that the most probable cause for the severe damage observed in industrial constructions during Cyclone Tracy in Australia in 1974 was fatigue failure in the vicinity of the connection holes. Mahendran (1990) performed fatigue tests with constant amplitude loads in "tangent arc" roof systems, in which the span length and the connection elements were taken as parameters in the study. Those tests were performed under constant amplitude repeated loads. Test with similar objectives were reported by Ellifrit and Burnette (1990) for cladding with trapezoidal cross section. Xu (1995) carried out tests on three types of metallic plates commonly used in Australia to evaluate the effect of cyclic loads of constant amplitude and loads which simulated the lifting of the roof due to negative pressures exerted by wind. The maximum reaction force at the screws was also investigated during the local crack propagation, and fatigue life predictions were made based on Miner's rule.

To investigate the failure capacity of the connection screws in metal system facades and metal roofs under static loads, Mahendran and Tang (1998) conducted full size and small scale experiments, in which they investigated the influence of secondary elements, thickness of plates, and yield limit, as well as the geometrical properties of screws. Based on those tests, Mahendran and Mahaarachchi (2002) concluded that, under sustained load fluctuations due to wind suction, the resistance of screws to failure was considerably reduced for few load cycles, and fatigue failure occurred for stress values lower than their static strength. It was also observed that two types of failures may occur around the connection holes: First, failure due to plate pull-through as a result of the dimpling of the plate in the vicinity of the connection hole, which allows it to detach through the head of the screw. Second, failure may occur as a result of screw pull-out, which is produced when the screw detaches itself from the secondary element to which it was connected. In the tests, secondary support elements of various thickness and steel grades were considered, investigating also the influence of the type of screw, its diameter, and the distance between cords.

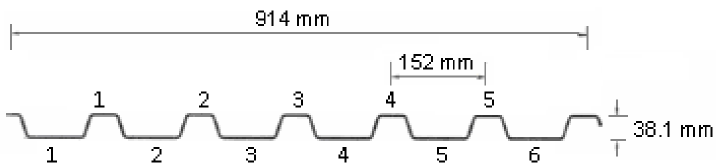
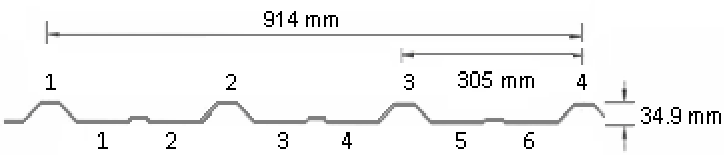
Research by Mahaarachchi and Mahendran (2004) concluded that the use of two-span specimens in the longitudinal direction with simple supports and one or two panels in the transverse direction are adequate when simulation of real behavior is attempted, whether it is investigated in a laboratory or through analytical models.

Recently, Mahaarachchi and Mahendran (2009) developed design formulae for local failures of crest-fixed trapezoidal steel claddings with closely spaced ribs under static uplift forces; the authors used experimental and analytical results and included advanced features such as the effect of geometric imperfections, buckling and hyperelastic behavior of neoprene washers. Henderson and Ginger (2011) validated the basis for the current cladding test standards that were derived using line-load tests. The authors used a pressure loading actuators to simulated real cyclonic loads. It was found that the effect of the spatial variation of wind loads on the reactions on a fastener is small. Yan and Young investigated the effects of temperature on screwed connections of thin metal sheets using steady-state (Yan and Young 2012a) and transient tests (Yan and Young 2012b). The authors found that the current specifications are conservative at elevated temperatures.

Other researchers have conducted full-scale tests to analyze failures induced by wind on metal roof corrugated plates. Figueroa (1996) carried out experiments on the cyclic capacity load of type B metal plates with connections in the valleys (see Table 1), which are commonly used as roof systems in industrial buildings in Puerto Rico since the landfall of Hurricane Hugo in 1989. In the experimental tests, two-span full-scale models were used. According to the predominant constructive practice, self-drilling connection screws, and various connection patterns (in successive and alternated patterns) were used. Four types of failures were examined during the tests and a formulation was developed for dynamic behavior under low and high fatigue cycles. The results showed that there is a 60% reduction with respect to the static load capacity in fatigue processes under high load cycles.

The current lack of empirical evidence on the behavior of screwed connections in roof systems and wall claddings with the characteristics found in the Caribbean region (i.e., geometrical configuration of the panels, elements and connection patters), indicates the need to perform new experimental tests in order to evaluate the fatigue resistance and the mode of failure of the connections when they undergo sustained load and unload cycles due to wind.

Table 1 Geometry of the decks used in the construction of roof and facades in industrial buildings (Matcor 2007). The numbers on top of the panels indicate crest screws, whereas numbers at the bottom indicate the positions of valley screws

Geometric configuration	Identification	Available gauges
	Type B Wide Rib (WR)	18-22
	Type E	24-26

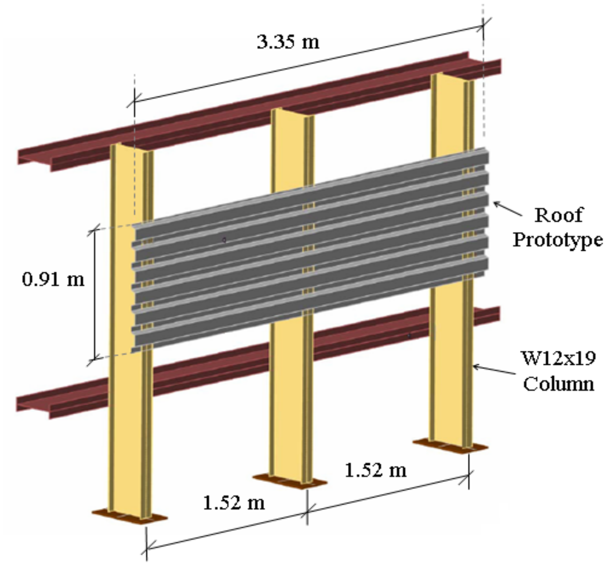


Fig. 1 Geometry of prototype used in present fatigue tests

The results of a research effort carried out to understand the fatigue behavior of steel claddings employed in the construction of industrial buildings are presented in this paper. The final objective of this research program was to produce fragility curves for this class of structures (García-Palencia *et al*. 2008), with the consequence that testing a large number of panels with different gauges (to cover for panels used in the construction in Puerto Rico and the Southern part of the United States) was considered more important than having accurate evaluations of the mechanics of behavior of the structural components. Section 2 deals with the experimental set-up used in the present work to test cold-formed plate systems under cyclic load. Because a fatigue protocol was required to perform the experiments, an adaptation of a protocol originally developed to account for fatigue of members constructed using non-metallic materials, which has proven to be an effective tool to rate wall cladding members under hurricane winds, was employed and is described in Section 3. Experimental results for various configurations are presented in Section 4, whereas discussion of results and conclusions are the subject of Section 5.

2. Experimental set-up

The characteristics of panels, connections, supporting frame, edge conditions, and load system, are described in this section. The panel and connection configurations were chosen according to information collected through field inspection and construction drawings of typical structures in Puerto Rico.

Panels

Metal panels employed in industrial buildings are commercially available in several configurations and gauges; in the course of this research, steel panels supplied by one manufacturer (Matcor 2007) were used and their dimensions are indicated in Table 1. Each panel

Fig. 2 Side view of test specimen, showing air bags. Panel Type E, gage 24

has dimensions of 3.35 m long by 0.91 m wide, as shown in Figs. 2-3. The first panel configuration is identified as Type B wide rib, which is one of the most commonly used in roof construction in Puerto Rico. This is available in gauges 18, 20 and 22, and is manufactured using grade 33 galvanized steel, with a yield stress of 230 MPa. A second configuration, identified as Type E, was also studied to take into account the performance of plates with low thickness (gauge 24); Type E has a different cross section, as shown in Table 1. Prototypes were tested to assess the fatigue capacity and to quantify the influence of employing washers on the fatigue behavior of panels in retrofitted configurations.

Connections

The panels were fixed to the supporting frame by means of self-drilling screws to three equally spaced W12 × 19 columns separated at 1.5m intervals, as shown in Fig. 1. Plate connections were made with self-drilling screws (N12 × 7/8" or 22.2 mm), with variations in the location and connection pattern. Constructive practices in Puerto Rico show a preference for using screwing connection systems in plate valleys, as shown in Fig. 3, with spacing between screws of 152 mm or 305 mm. Likewise, there are systems with connections on the crests of the plate (see Fig. 4); this constructive practice turns out to be more convenient in terms of drainage, especially in the roof. In typical daily constructions scenarios in Puerto Rico, cladding holes are drilled in situ before installing the screws. Consistently, in the lab, holes were drilled in the specimens by an expert technician who had experience in the construction industry.

The authors did not compare the adopted installation with alternative forms of performing the task, which may be representative of construction practices in other countries. The improvement alternative proposed in conventional systems with connections in the valleys consist on increasing the connection area between the head of the screw and the metallic plate through the addition of washers, which are shown in Fig. 5. In this way it is possible to decrease the stress concentrations around connection holes, which are directly related with the appearance of one of the typical failure mechanisms in this type of systems characterized by pull-through of the metallic plate. The

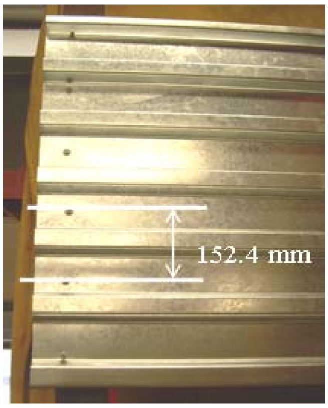


Fig. 3 Test specimen with connections in valleys. Panel Type B, gage 22

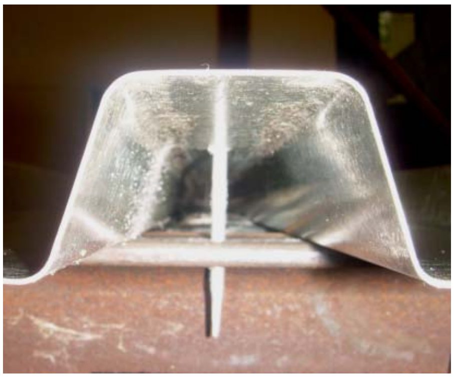


Fig. 4 Test specimen with connections in crests. Panel Type B, gage 22

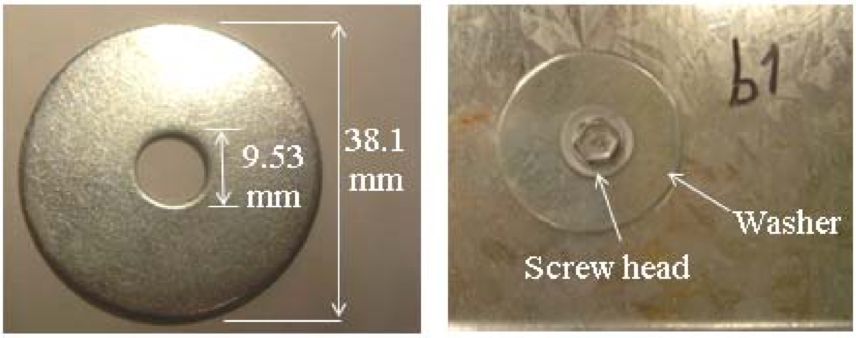


Fig. 5 Details of washer and improved connection

washers used have an internal diameter of 9.5 mm and 38.1 mm external diameter; these are placed under the neoprene washers that come included in the screws and are used as required by the connection patterns. During the installation process of the screws, special precautions were taken: (1) The screws were tightened until the neoprene washers allowed it, to avoid local buckling in the plate; (2) in the case of connection in the crests (Fig. 4), the holes were pre-drilled to ensure the least possible harm to the plate, and (3) it was ensured that the longitudinal axis of the screws remained perpendicular to the plane of the plate.

Supporting frame

The testing facility, shown in Figs. 1-6, consists of a single span support frame, which holds two W6×12 steel beams. Three W12×19 columns were employed to provide support to the test specimens, which were set with high resistance bolts to one steel plate of 12.7mm thickness and which in turn connect to the floor by means of two anchoring bolts. It was welded to the base of each column mainly to transfer the shear generated during the test. The steel frame has lateral bracing outside its plane to avoid out-of-plane displacements during the test. A short column at the center of the frame provides additional vertical support to the beams; the load plate located in the rear end is reinforced by a rail mechanism on each side, which at the same time, rests on an auxiliary frame.

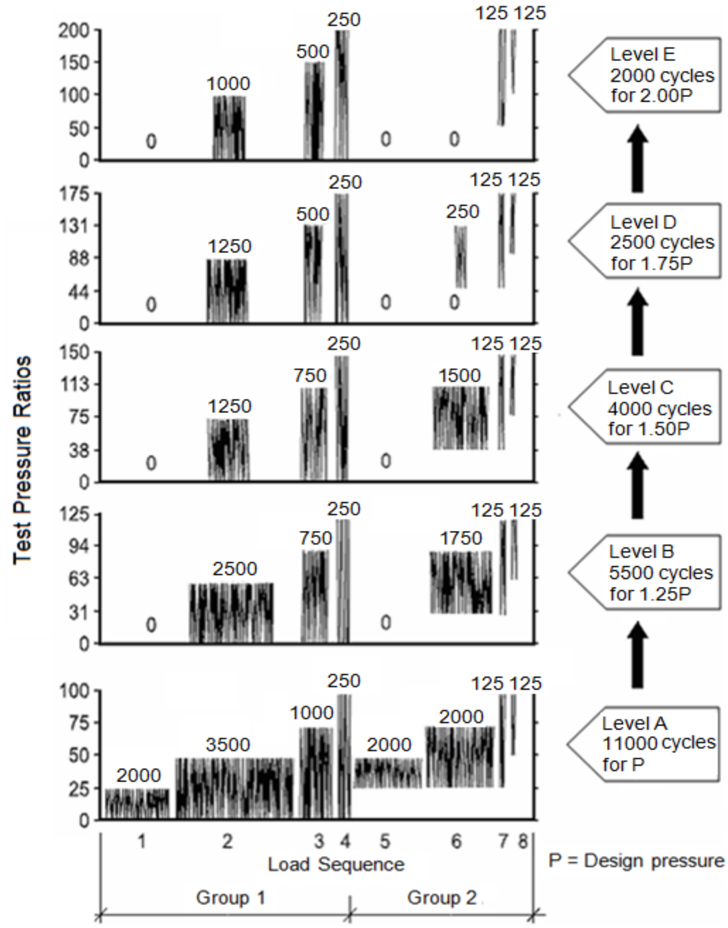


Fig. 6 Dynamic protocol SIDGERS-5 (Adapted from Baskaran *et al*. 1999)

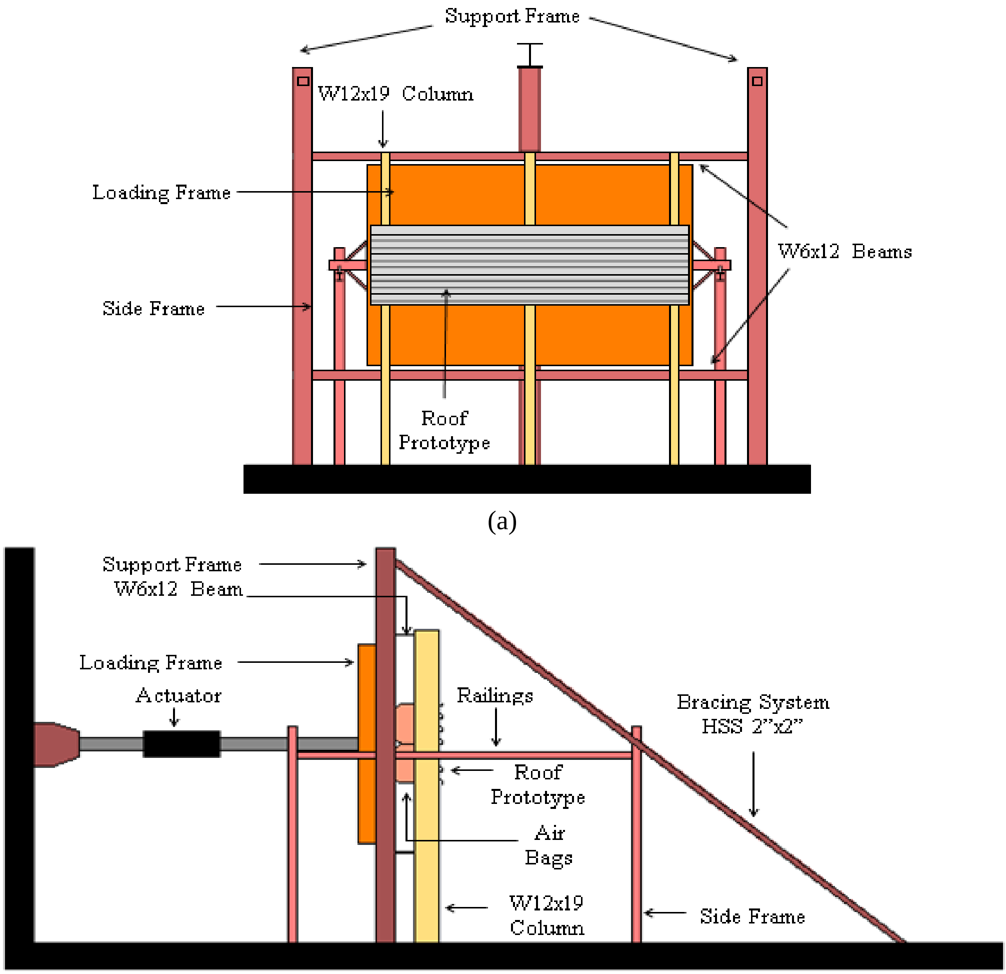
Edge conditions

The deflections at the sides of the panel were not restrained because previous tests indicated that the panels always failed at the internal connections. The authors acknowledge that the rotation of the cladding rib at one edge might have an effect on the results; this could not be quantified.

Load system

To generate the load, an MTS hydraulic load actuator was used, with a digital function generator being employed to simulate the effect of suction produced by wind as a sinusoidal function. The actuator is capable of applying a maximum load of 160.14 KN and a maximum displacement of 25.4 mm. To protect the integrity of the test frame, the specimens were subjected to loads that did not exceed 26.7 KN. The load sequence shown in Fig. 7 was applied as an equivalent pressure on each configuration. Uniform pressure distribution of the hydraulic jack loads to the test prototypes was done by means of twelve cubic air bags, manufactured with vinyl, which were located between the load plate and the metal panel. The air bags were protected with layers of adhesive tape and reinforcing tape to guarantee that they resisted the local loads during the test, they adhere to the metallic panel with Velcro® strips (Fig. 6). A comment must be made regarding the way the load was applied during the tests. The use of air bags for making an analogue of wind loading may have some drawbacks, as the air bag does not make complete contact with the panel (Henderson 2010), inducing additional forces in the specimen (Seccombe *et al*. 1996).

The configurations that were tested in this research are indicated in Table 2. The numbering of the tests corresponds to the order in which they were made, and only those of fatigue are reported in this paper. The deck gauge is indicated in each case, namely gauge 22 (thickness = 0.76 mm), gauge 20 (t = 0.91 mm), or gauge 18 (t = 1.21 mm). The location of the screws is indicated as a sequence of numbers and is shown in Table 1. A typical panel Type B has six valleys and five crests. For configurations with screws in valleys, the notation 1-2-3-4-5-6 indicates that all valleys had a connecting screw. In the 1-3-4-6 configuration, there were screws in all valleys except for locations 2 and 5.



(b)
Fig. 7 Test apparatus: (a) Front view; (b) Side view

3. Load protocol adopted in this research

Over a decade ago, the SIDGERS group in Canada proposed the use of a dynamic load protocol to evaluate fatigue behavior, which was originally designed to evaluate polyvinyl chloride (PVC) and ethylene-propylene-diene-monomer (EPDM) membranes used as waterproofing for roof plates (Baskaran *et al*. 1999, Baskaran *et al*. 2012). Unlike in other procedures, in SIDGERS one can choose a reference load which is not necessarily the expected load at failure, and the fatigue test goes through several levels, each with blocks of equal amplitude oscillations. One of the advantages of the concept behind SIDGERS is that it allows the development of a detailed classification (rating) of a specimen by considering the last complete level that was passed without failure. Also, the protocol accounts for variations explicitly codified in North American wind standards (Baskaran *et al*. 1999), such as the effects of exterior wind fluctuations combined with the constant interior pressure over the building.

Based on SIDGERS, Avilés (2006) proposed the use of a modified dynamic load protocol (called SIDGERS-5) to evaluate the fatigue capacity of wood and zinc roofs. This modified protocol was also employed by García (2007). Due to the higher resistance of metals with respect to the materials considered for the original version of the protocol, the tests conducted by Avilés showed the need to maintain a number of low fatigue cycles of 10,000, in order to be consistent with the TR440 protocol (Experimental Building Station 1978) that has proven to be adequate in the assessment of the behavior of metallic panels. The solution proposed and subsequently verified, consists in multiplying the number of cycles in all levels by five. In other words, Level A rises from 2,200 to 11,000 cycles, Level B from 1,100 to 5,500, and so on. In this way, if a sample completes level A, it is assigned a load value P to obtain its classification corresponding to the first level. If the same sample next completes level B, then it is given a 1.25 P classification.

The procedure is schematized in Fig. 7, and includes eight sequences of load (which represent wind gusts), to which the components are subjected. The load sequences are grouped in five different levels, identified as Levels A to E. There are two groups of cycles with which the tests are conducted: Cycles of Group 1, which represent simulated suction induced by wind over the roof system, and cycles of Group 2, which simulate the effects of exterior wind fluctuations combined with the constant interior pressure over the building. Each group consists of four sequences where pressure levels are oscillated between zero and a fixed pressure.

Each one of the sequences is applied with a pressure that is a percentage of the wind pressure design stipulated by design and construction codes for a given type of structure and a geographic location, starting with low pressures and increasing them as the level changes. For example, the test in level A includes a sequence of 2,000 cycles (gusts) at 25% wind design speed, another sequence of 3,500 cycles at 50% wind design speed and so on, until 11,000 cycles are completed. If a sample can complete all five levels, it can reach a classification twice as high as the one used on level A.

The main advantage of a protocol like SIDGERS with respect to other load protocols lies in its economy, since with a single sample one can reach various classifications. However, this load protocol cannot be directly used in panels constructed with other materials, because the number of cycles required to fail under fatigue loads depends on the material properties.

In the original SIDGERS protocol only five levels, named A-E, were available; however, more levels are often required for metal decks. For any level greater than E, the same sequence as in level E is used but the maximum load is increased by 0.25 on each subsequent level until the first connection failure is observed (Aviles 2006). For example, level F has the same number of cycles and the same sequence as level E but with a maximum load P = 2.25.

(a) (b) (c)

Fig. 8 Examples of failure modes: (a) Connections in valleys; (b) Connections in crests; and (c) Improved connections in valleys

4. Experimental results

A total of nineteen full-scale specimens were tested under the load protocol of Fig. 7 described above. In most cases, three types of connections were investigated for each deck configuration, including crest and valley connections reinforced by washers.

A definition of failure is required in this protocol in order to stop the test and give a rating to the specimen. Because of the need to protect the testing equipment, and at the same time to maintain the load uniformly applied, the specimens were tested up to a point in which three connections of central support failed. In general, at this load level the remaining screws are not able to hold additional loads transferred to them because of the loss of the first connections.

In general, the tests conducted over similar specimens resulted in similar results, with percentage differences that do not exceed 5% and in any case, the lowest resistance value was chosen. The failure mode observed in all the tested configurations corresponds to tearing of the metal plate due to spreading of cracks as a result of fatigue in the vicinity of the connection holes. Failure in the form of metal plate pull-through caused by the initiation and propagation of fatigue cracks (both longitudinal and diamond-shaped) in the vicinity of the connection holes was the predominant failure mode observed during the tests (see Fig. 8). Failure is located at the connections in the central support of Fig. 1, because the reaction force in the support is high as a consequence of its larger influence area. In the present experimental program, each configuration had different fatigue strength, so that the basic load P chosen as a reference load in most cases is different. The results are summarized in Table 2, and are discussed in the remaining of this section.

4.1 Results for systems with connections at the valleys

4.1.1Type B wide rib

Nine specimens with connections located in valleys were tested. Test 1 initiated with a relatively large load of 13.3KN, with the consequence that failure was not reached and the results could not be completed.

Two further tests (Test 2 and 3) were performed on the same configuration (305 mm between screws, or 1-2-3-4-5-6, gauge 22), but with a lower load P = 8.9 kN; both provided exactly the same rating of 13.3 kN, with the last connections failing at levels D4 and D6 respectively. In terms of the SIDGERS protocol, the specimens completed the C rating with a basic load P = 8.9 kN and a force of 1,112 N per connection which is equal to the reaction at the middle support divided by its number of connections. Test 4 was a repetition of Test 1, but this time with a load reduced to P = 4.4 kN, spacing = 305 mm, gauge 22. For this weaker configuration, the specimen completed level E with a load rating reduced to 8.9 kN, but the resistance of each connecting screw remained the same (1,112 N). Last failure was reached at level F4 of the protocol.

Test 15 and 16 are the same configuration, with gauge 20, spacing = 305 mm, with an initial load of 6.7 kN. The maximum load rating was 10 kN and 1,250 N/connection. The specimen rated C in the fatigue protocol.

Finally, thicker plates with gauge 18 were tested in Tests 13 and 14, with spacing = 305 mm. Fatigue rating was E with initial load 6.7 kN, reaching a maximum load of 13.3 kN, and 1,668 N/connection.

In summary, as the plate thickness increases from gauge 22, to 20, and to 18 (Tests 4, 15, and 13), the maximum loads increased from 8.9 kN (completed E with initial load 4.4 kN), to 10 kN (completed C with initial load 6.7 kN), and to 13.3 kN (completed E with initial load 6.7 kN), respectively. To account for the number of screws used in connections as a consequence of a reduction in spacing from 305mm to 152.4 mm for the same gauge 22 (Tests 4 and 2), the load increased from 8.9 kN (completed E with initial load 4.4 kN), to 13.3 kN (completed C with initial load 8.9 kN); however, the load taken by each connection at failure remained the same in both cases (1,112 N).

Table 2 Definition of test configuration and results: Screws locations are indicated in Table 1. Column 6 indicates the load level in the SIDGERS-5 protocol at which failure of the first screw occurred

Test	Deck gauge	Deck Type	Screw Locations (Table 1)	Initial applied load [kN]	First screw failure	Rating: Completed protocol sequence	Rating: Maximum Load [kN]	Load at failure [N/connection]
1	22	B	1-3-4-6 (*v)	13	A2	A3	10	---
2/3	22	B	1-2-3-4-5-6 (*v)	8.9	B4	D4	13.3	1112
4	22	B	1-3-4-6 (*v)	4.4	F3	F4	8.9	1112
15/16	20	B	1-3-4-6 (*v)	6.7	C	D4	10	1250
13/14	18	B	1-3-4-6 (*v)	6.7	E4	F3	13.3	1668
23	24	E	1-2-3-4-5-6 (*v)	6.7	C	D4	10	836
19	18	B	1-2-3-4-5 (*c)	4.4	N	---	---	---
21	22	B	1-3-5 (*c)	5.3	D3	E4	9.3	1557
20	20	B	1-3-5 (*c)	5.3	D3	G4	12	2002
24	18	B	1-3-5 (*c)	5.3	G4	H4	13.3	2224
27	24	E	1-2-3-4 (*c)	4.4	D4	E3	7.8	974
5	22	B	1-2-3-4-5-6 (*vw)	8.9	H	I3	24.5	2037
6	22	B	1-3-4-6 (*vw)	8.9	F3	F8	17.8	2224
17	20	B	1-3-4-6 (*vw)	6.7	I4	J4	20	2504
18	18	B	1-3-4-6 (*vw)	11.1	G8	--	---	---
26	24	E	2-3-4-5 (*vw)	5.4	F4	G3	13.34	1668

* v: screws are located in valleys; *c: screws are located in crests; *vw: screws are located in valleys with washers. Locations details are presented in Table 1

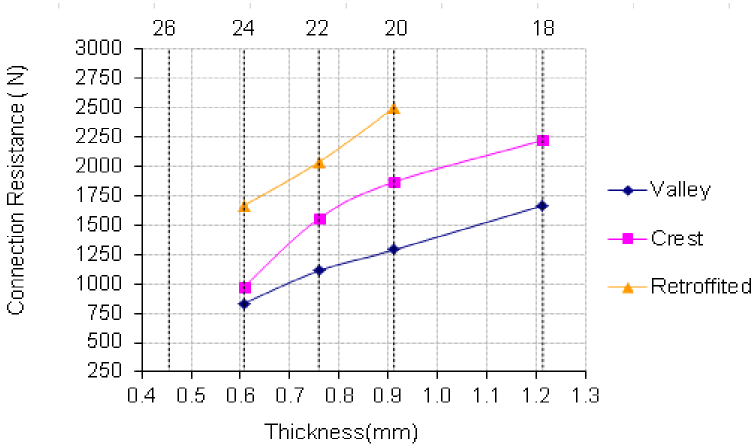


Fig. 9 Connection strength as a function of plate thickness

4.1.2 Type E

Only one specimen with gauge 24 was tested (Test 23) and corresponds to a panel with spacing of 305 mm and initial load 6.7 kN. The first failure occurred at C and the last one at D4, for a rating equal to 10 kN or 836 N per connection.

4.2 Results for systems with crest connections

4.2.1 Type B wide rib

The separation between connections was fixed at 305mm in three tests (Tests 21, 20, and 24) with different gauge (22, 20, 18), reaching 9.3 kN with 1557 N/connection; 12 kN with 2,002 N/connection; and 13.3 kN with 2,224 N/connection, respectively. The general trend was as expected, with loads increasing with gauge. Fatigue rating was D, F and G respectively.

For the same gauge and spacing, comparisons can be made between valley and crest connections. For example, for gauge 22 valley-fixed specimens, with spacing of 305 mm (Test 4) rated 8.9 kN with 1,112 N/connection, whereas crest-fixed specimens (Test 21) rated 9.3 kN with 1,557 N/connection.

A valley-fixed specimen with gauge 20, spacing 305 mm (Test 15) rated 10 kN with 1,250 N/connection; whereas the equivalent crest-fixed configuration of Test 20 rated 12 kN with 2,002 N/connection. For gauge 18, valley connections with spacing of 305 mm (Test 13) rated 13.3 kN with 1,668 N/connection; crest connections in Test 24 rated 13.3 kN with 2,224 N/connection. In the three cases compared, the rating loads were similar and only gauge 20 showed differences.

Because fewer screws are used in crest connections than in valley connections, the load resisted by each connection was higher in crest-fixed specimens.

4.2.2 Type E

One specimen (Test 27) with spacing 152.4 mm was tested. The initial load was set equal to 4.4 kN; First failure occurred at sub-level D4 and the last one at E3. This configuration rated 7.78 kN or 974 kN/connection.

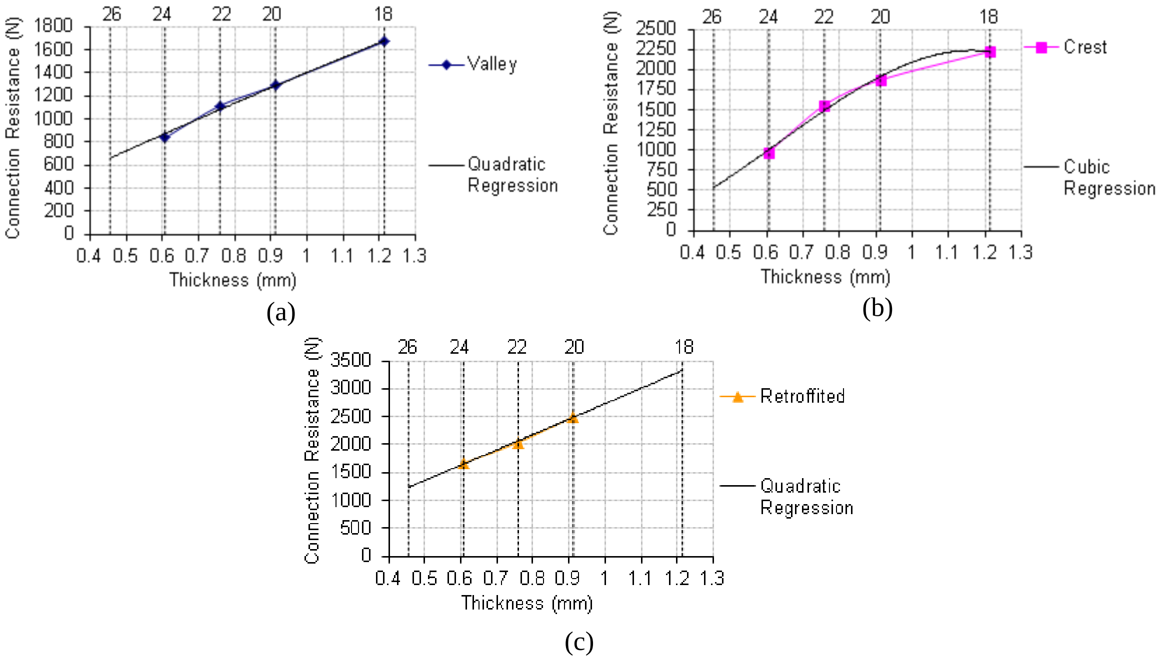


Fig. 10 Polynomial regressions for the strength of the tested prototypes: (a) Connections in Valleys; (b) Connections in crests; (c) Improved connections

4.3 Results for System with improved connections

4.3.1 Type B wide rib

In valley-fixed configurations, a dramatic increase in maximum load was observed. For gauge 22, with spacing 152.4 mm, the load increased from 13.3 kN (Test 2) to 24.5 kN (Test 5), whereas for spacing 305 mm, the increase was from 8.9 kN (Test 4) to 17.8 (Test 6).
For 305mm spacing, comparisons can be made for gauge 20, which changed from 10 kN (Test 15, conventional connection) to 20 kN (Test 17, improved connection).
The proposed improved alternative provides a considerable increase in the connection performance, with resistance increasing between 175% and 200% of that obtained in conventional valley-fixed systems.

4.3.2Type E

Similar to the results obtained for Type B wide rib, the resistance increased from 836 N/connection in the conventional configuration (Test 23) to 1,668 N/connection (Test 26) and gauge 24.
Although there is variability in the actual value of the load at which failure occurs (as in any fatigue testing), the classification of each panel did not show variability: for example, Tests 2 and 3, which were performed on the same panel configuration, provided the same classification; and the same thing happened for Tests 13 and 14, and for Tests 15 and 16. As in any fatigue experiment it would have increased the level of certainty if several tests would have been carried out for each configuration and gauge; this however was not possible within the budget limitation of the authors.

Table 3 Values of A, B, C and R^2 corresponding to the proposed polynomial regressions between the strength of the connections and the deck thickness for the tested specimens

Connection	A	B	C	R^2
Valley-fixed	0	-88.601	1487.9	0.993
Crest-fixed	-3818	7234	-1328	0.992
Connection with washer	0	40.089	2693.7	0.996

4.4 Summary of results

Fig. 9 shows the variation of fatigue resistance for each type of connection obtained in the tests, as a function of the thickness of the plate. As will be discussed in the following paragraphs, some data could not be obtained and it was necessary to either extrapolate or interpolate between experimental points.

For each one of the prototypes considered, polynomial regression was conducted to obtain a cubic equation of the form

$$F = At^3 + Bt^2 + C \tag{1}$$

where F represents the connection resistance (N), and t is the thickness of the plate (mm); the corresponding plots in SI units are shown in Fig. 10. The values of the coefficients A, B, C and correlation coefficients R^2 are listed in Table 3. The correlation coefficients obtained from the polynomial regression are greater than 0.99, which indicates a good fitting with respect to the proposed type of regression. Taking into account the regression equations of each prototype studied, the next term is the extrapolation of the missing resistances.

This empirical equation of connection resistance versus plate thickness was used to estimate missing values not obtained during the experiments. Specifically, the equation was used for two missing pieces of data: First, in the case of the gauge 18 improved connections, it was not possible to obtain a resistance value, because the extremely high load level required for inducing a failure could compromise the stability of the testing equipment. Second, the resistances corresponding to specimens of gauge 26 in all connections were obtained by data extrapolation.

5. Conclusions

A modified dynamic load protocol has been employed to assess the fatigue capacity of steel cladding used as wall and roof in industrial buildings. This protocol is a modification of an earlier one developed by a research group in Canada, known as SIDGERS. One of the objectives of the present research was to evaluate the feasibility of using SIDGERS to rate steel decks used as cladding in order to simulate behavior under wind. The result of 19 tests carried out by the authors indicates the adequacy of the protocol in distinguishing between different configurations of deck thickness and connection types on fatigue behavior.

The second objective was to quantify the relative strength of various configurations. For example, increasing the plate thickness is expected to increase fatigue strength; however, finding the quantitative increase is a contribution of this paper. Because the fatigue capacity of panels has been obtained by means of carefully conducted experiments, it is expected that they may serve other researchers in calibrating numerical models of this problem.

It is important to note that if the load of the initial test is close to the capacity of the specimen, the protocol tends to overestimate the real system classification, as it happened in the first test conducted. For this reason, it is recommended to initiate the tests with low values of loads, although this could result in longer testing times.

Finally, the authors were not able to follow the pattern of stress redistributions which takes place in the plate following first screw failure. What the authors measured is the overall load level between failure of one screw and failure of three screws, and in reaching this overall capacity there are a number of redistributions that take place. Following such stress redistributions was out of the scope of this work, although it would provide valuable information for the mechanics of behavior of this type of structures.

Acknowledgements

The authors thank Prof. Ali Saffar for his valuable contribution during the development of this work. Testing was done at the Structures Laboratory, Department of Civil Engineering, University of Puerto Rico at Mayaguez. The second author is a member of the scientific staff of CONICET (Science and Technology Research Council of Argentina). This research was partially supported by grants from the Insurance Commissioner in Puerto Rico and from CONICET and SECYT-UNC in Argentina.

References

Avilés, D. (2006), "On improving the performance of wood-zinc roof system", PhD Dissertation, University of Puerto Rico, Mayagüez, Puerto Rico.

Baskaran, B.A., Chen, Y. and Vilaipornsawai, U. (1999), "A new dynamic wind load cycle to evaluate flexible membrane roofs", *J. Test. and Eval.*, **27**(4), 249-265.

Baskaran, A., Ham, H. and Lei, W. (2006), "New design procedure for wind uplift resistance of architectural Metal Roofing Systems". *J. Arch. Eng., ASCE*, **12**(4), 1192-1201.

Baskaran, A., Molleti, S., Ko, S. and Shoemaker, L. (2012), "Wind uplift performance of composite metal roof assemblies", *J. of Ach. Eng., ASCE*, **18**(1), 2-15.

Beck, V. and Stevens, L. (1979), "Wind loading failures of corrugated roof cladding", *Civil Eng. Trans.*, Institution of Civil Engineers (Australia), **12**(4), 168-177.

Cook, N.J., Keevil, A.P. and Stobart, R.K. (1988), "BRERWULF-The big bad wolf", *J. Wind Eng. and Ind. Aerod.*, **29**(1), 99-107.

Experimental Building Station (1978), "TR 440: Guidelines for Testing and Evaluation of products for Cyclone prone areas", Experimental Building Station, Sydney, Australia.

Ellifritt, D.S. and Burnette, R.I. (1990), "Pull-over strength of screws in simulated building test", *Proceedings of the 10th Int. Specialty Conf. on Cold-Formed Steel Structures*, St. Louis, MO, USA, 589-603.

Factory Mutual Research (1988), "Approval standard: Class I roof covers (4470)", Norwood, Massachusetts.

Figueroa, V. (1996), "Performance of Steel Roof Decks Subjected to Repeated Wind Loading", MS Thesis, Universidad de Puerto Rico, Mayagüez, Puerto Rico.

García, R. (2007), "Development of hurricane-based fragility curves for wood-zinc houses in Puerto Rico", PhD Dissertation, University of Puerto Rico, Mayagüez, Puerto Rico.

García-Palencia, A.J., Saffar, A. and Godoy, L.A. (2008), "Curvas de fragilidad debidas a viento para edificaciones industriales metálicas", *Rev. Int. Des. Nat., Accid. Infr. Civ.*, **8**(2), 165-182.

Gerhardt, H.J. and Jramer, C. (1988), “Wind loading and fatigue behavior of fixings and bonding of roof coverings”, *J. Wind Eng. Ind. Aer.*, **23**(1), 243-247.

Henderson, D.J. (2010), “Response of pierced fixed metal roof cladding to fluctuating wind loads”. PhD Dissertation, James Cook University, Townsville, QSL, Australia.

Henderson, D.J. and Ginger, J.D. (2011), “Response of pierced fixed corrugated steel roofing systems subjected to wind loads”, *Eng. Struc.*, **33**(12), 3290-3298.

Henderson, D.J., Ginger, J.D., Morrison, M.J. and Kopp, G.A. (2009), “Simulated tropical cyclonic winds for low cycle fatigue loading of steel roofing”, *Wind Struct.*, **12**(4), 381-398.

Kopp, G.A., Morrison, M.J., Gavanski, E., Henderson, D.J. and Hong, H.P. (2010), “Three little pigs” project: hurricane risk mitigation by integrated tunnel and full-scale laboratory Tests”, *Nat. Haz. Rev., ASCE*, **11**(4), 151-161.

López, H.D. and Godoy, L.A. (2005), “Metodología para la estimación de daños estructurales ocasionados por vientos huracanados en edificaciones industriales”, *Rev. Int. Des. Nat., Accid. Infr. Civ*, **5**(2), 121-134. (in Spanish)

Mahaarachchi, D. and Mahendran, M. (2004), “Finite element analysis and design of crest-fixed trapezoidal steel claddings with wide pans subject to pull-through failures”, *Eng. Struc.*, **26**(1), 1547-1559.

Mahaarachchi, D. and Mahendran, M. (2009), “Wind uplift strength of trapezoidal steel cladding with closely spaced ribs”, *J. Wind Eng. Ind. Aerod.*, **97**(1), 140-150.

Mahendran, M. (1990), “Fatigue behavior of corrugated roofing under cyclic wind loading”, *Civ. Eng. Trans.*, Institution of Engineers, Australia, **32**(4), 219-226.

Mahendran, M. and Mahaarachchi, D. (2002), “Cyclic pull-out strength of screwed connections in steel roof and wall cladding systems using thin steel battens”, *J. Struc. Eng., ASCE*, **128**(6), 771-778.

Mahendran, M. and Tang, R.B. (1998), “Pull-out strength of steel roof and wall cladding systems”, *J. Struc. Eng., ASCE*, **124**(10), 1192-1201.

Matcor (2007), *Rolled Formed Metal Building Components*, Matcor Inc., Guaynabo, Puerto Rico, USA.

Morgan, J. and Beck, V. (1977), “Failure of sheet metal roofing under repeated wind loading”, *Civ. Eng. Trans.*, Institution of Engineers, Australia, CB19, 1-5.

Newman, A. (2003), *Metal Building System: Design and Specifications*, (2nd edition), McGraw-Hill, New York NY, USA.

Norwegian Building Research Institute (1990), “NBI 162-90 Roof Coverings: Dynamic wind-load resistance”, Norwegian Building Research Institute, Trondheim, Norway.

Seccombe, C.J., Carne, S.J. and Martin, B.A. (1996), “Improvements in system evaluation under static and cyclic wind loads”, *National Engineering Conference: Engineering Tomorrow Today*, Institution of Engineers, Darwin, NT, Australia, 33-38.

Underwriters Laboratories (1996), “Test for uplift resistance of roof assemblies”, Standard UL 580, Northbrook, IL.

Xu, Y.L. (1995), “Fatigue performance of screw-fastened-light-gauge-steel roofing sheets”, *J. Struc. Eng., ASCE*, **121**(3), 389-398.

Yan, S. and Young, B. (2012a), “Screwed connections of thin sheet steels at elevated temperatures – Part I: Steady state tests”, *Eng. Struct.*, **35**(1), 234-243.

Yan, S. and Young, B. (2012b), “Screwed connections of thin sheet steels at elevated temperatures – Part II: Transient state tests”, *Eng. Struct.*, **35**(1), 228-233.