16th World Conference on Earthquake Engineering, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 293

Registration Code: S-G1464738152

SEISMIC PERFORMANCE OF WEAK-BASE STRONG COLUMN STEEL MOMENT FRAMES

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Abstract

Steel moment frames in seismically active regions are designed as strong-base weak-column systems, such that yielding is concentrated in the lower region of the column member, thereby protecting the base connection itself. This is achieved through capacity design of the base connection. This necessitates an expensive base connection, since heavy detailing is required to develop the forces associated with column yielding. However, recent research by the authors and others has shown that column base connections may be highly ductile with excellent dissipative characteristics. The research explores the possibility of a weak-base system that leverages these characteristics. The study is conducted in a Performance Based Earthquake Engineering (PBEE) framework, by examining the effect of base strength on various performance metrics – including probabilities of collapse, and of exceeding critical member forces and interstory drifts. The results indicate promise for these types of systems, but also highlight issues, which arise due to reduce base stiffness that accompanies the weaker base. Limitations of the study are discussed along with directions for future work.

Keywords: Baseplates; Column base connection; Performance-based design; Seismic demand assessment

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1. Introduction

The basic intent of building code seismic provisions is to provide buildings with the ability to withstand intense ground shaking without collapse, however, some significant structural damage is allowed. In order to accomplish this, the basic principle is to encourage the use of building configuration, structural system and material, and component detailing for a global resilient behavior. Within this setting, Steel Moment Resistant Frames (SMRFs) are one of the most common and popular lateral load resistant systems used in seismic regions due to their architectural versatility and energy dissipative properties. In low- to mid-rise SMRFs, Exposed Column Base (ECB) connections are often used to connect the steel column to the concrete foundation. These connections are used in gravity frames to resist axial tension or compression, and in moment frames to resist flexure and shear, along with axial force. In essence, this necessitates an expensive base connection since heavy detailing is required to develop the forces associated with column yielding. In the current design practice, ECB connections are designed to resist the maximum capacity of the attached columns (i.e. $1.1R_yM_p$), based on the capacity design principle, implying a "strong base weak column design". This approach leads to the development of the full flexural capacity of the columns and ultimately to the formation of plastic hinges on them, with the presumption that a plastic hinge within the member will have higher rotation capacity. However, experimental data curated by Lignos and Krawinkler (2007) [1] disaffirms this, suggesting that the ductility of plastic hinges in columns may be compromised by phenomena such as local or lateral torsional buckling. In large measure, the reliance on strong bases may be attributed to: (1) the general notion that connections are less ductile than members, and (2) the lack of system simulation or testing that demonstrates acceptable performance with weak-base systems.

Numerous experimental and analytical studies have been conducted on ECB connections to establish their strength, stiffness, deformation capacity, and failure modes. Experimental studies by DeWolf and Sarisley, 1980 [2]; Astaneh et al., 1992 [3]; Burda and Itani, 1999 [4]; Fahmy et al., 1999 [5], and more recently Gomez et al., 2010 [6], and Kanvinde et al., 2014 [7], have resulted in quantitative understanding of various aspects of ECB response. These experimental investigations have demonstrated that if properly designed and detailed, ECB connections can provide an inelastic rotation capacity in the range of 0.1 radians. Also, these tests revealed that along with this significant rotation capacity, base connections themselves are able to substantially dissipate hysteretic energy. These experiments (along with analytical models) suggest that even if designed as strong bases, ECB connections exhibit significant flexibility. However, in terms of computational modelling they are typically simulated as fixed, resulting in unrealistically optimistic estimates of building response in terms of member forces, story drifts, and the likelihood of collapse. More specifically, rotational flexibility of ECB connections concentrates deformations in the first story of the building, greatly diminishing ductility compared to the design assumption [8].

The current design methods disregard the excellent dissipative features of ECB connections, leading to an increase in the price of fabricating them. The experimental results obtained from tests revealed that the approach of designing the base connections as yielding elements is promising and would lead to desirable economic savings and to a more accurate characterization of the building response. Motivated by the preceding discussion, the main objective of this paper is to contribute to the characterization and validation of a methodology for the design of SMRFs with "weak-bases strong-column". For this purpose, the study investigate the seismic performance of a four story building with exposed base connections with respect to the variation in their strength. This kind of connection is the most common form of anchoring system for low and mid-rise buildings, including commercial, residential and industrial structures. Thus, the analysis of this type of base connection enables the general characterization of the proposed approach for numerous and common building structures. The four story building was selected to be a general representation of a low or mid-rise building. Also, four stories would generally be the limiting height for the use of exposed connections.

The description of the seismic performance of the structure was primarily accomplished by means of Multiple Stripes Analysis (MSA). As described by Jalayer and Cornell (2009) [9], this nonlinear analysis procedure enables performance studies to test numerous performance targets, including collapse prediction, for a wide variety of ground motions. The ground motion records used to support the MSA in this investigation are selected from the Applied Technology Council's ATC-63 project (FEMA 2009) [10]. In total, 32 ground motions were used, only including far-field records. In order to evaluate the effect of base strength in the framework of Performance based Earthquake Engineering (PBEE), five connection strength levels were investigated. They include a fixed connection, which would represent the assumption taken by the current design codes for a building as such; a Pinned connection, and three connections whose response is characterized in terms of a hysteretic model and which vary their strengths from one, representing a connection designed to the capacity of the attached column, to 0.5 and 0.3 times the former.

2. Background

In the context of earthquake engineering, SMRFs are designed to resist earthquake ground shaking based on the assumption that they are capable of extensive yielding and plastic deformation without extensive deterioration. The intended plastic deformation consists of plastic rotations developed within the beams, at their connections to the columns. Damage is expected to consist of moderate yielding and localized buckling of the steel elements. SMRFs are anticipated to develop their ductility through the development of yielding at the beam column connections. This yielding may take the form of plastic hinging in the beams (or, less desirably, in the columns), plastic shear deformation in the column panel zones, or through a combination of these mechanisms. Beam to column connections used in SMRFs shall be capable of accommodating a story drift angle of at least 0.04 rad. Regarding to column base connections, current design provisions (i.e. Seismic Provisions AISC 341-10, 2010 [11]) encourage a designed based on a "strong-base weak-column criteria", such that yielding is concentrated in the lower region of the column member, thereby protecting the base connection itself. Taking this into consideration, failure of the base connections should rarely occur. However, and as pointed out by Aviram et al. (2010) [12], ECB connection failures have been observed in a large number of SMRFs during the 1989 Loma Prieta, 1994 Northridge and 1995 Kobe Earthquakes. Motivated by these unexpected failures, many studies have been targeted to investigate the effect that the mechanical properties of ECB connections cause on building response and performance.

The response of ECB connections is determined by a complex interaction between its parts. These include contact and gapping between the plate and the grout, yielding of the bolts, crushing of the grout and yielding of the plate itself. The axial forces are transmitted directly to the base plate through the entire/effective cross sectional area of the column. The lateral forces on to which the building could be subjected would create differential stress profiles under the base plates depending on variables such as plate thickness or the relative stiffness between the plate and the supporting material [13]. Primarily, the studies conducted on ECB connections have focused on identifying strength limit states of these connections and providing approaches for characterizing the strength of base connections culminating in the development of design guidelines such as the American Institute of Steel Construction's (AISC's) *Design Guide 1* [14]. The current design approach (AISC Steel Design Guide One) assumes that the stress bulb takes the form of a rectangular block. It is important as well to describe the failure modes that this kind of connections may exhibit. Gomez et al. (2010) [6] define three common failure modes assumed in design. They are: (1) anchor rod yielding in tension due to uplift of the plate; (2) the base plate reaches its tensile capacity due to the forces exerted by the anchor rods pulling; and (3) yielding due to the bending produced by the bearing stresses between the plate and the concrete.

A comprehensive method for determining the rotational stiffness of ECB connections was proposed by Kanvinde et al. (2012) [15]. This methodology allows to account in simulations for the dissipative properties that base connections possess. The method is derived from existing design procedures in order to simplify the calculation of the rotational stiffness. The method consist basically in three steps: 1) Characterization of design strength (M_y) of the connection in accordance with current design procedures. 2) Characterization of deformation of individual components on the basis of internal force distribution, and 3) Enforcement of compatibility over the various components to determine connection rotation at the applied base moment. Its accuracy depends on the moment to axial load ratio applied, being particularly accurate for high ratios and overestimating for low ratios. This approach enabled the possibility of investigating the seismic response of buildings with respect to base stiffness. Zareian and Kanvinde [8] studied this particular effect through a series of parametric sophisticated simulations for a wide arrange of structures noting that overestimating the fixity of base connections (or simulating the base connections as fixed) results in significant detriment to building performance, including increased interstory drift and collapse probability. Consequently, the characterization of the flexibility of the connections is critical to predict its effectiveness, and their influence in SMRF at a global level. Thus, it serves as a starting point to begin evaluating the influence of variation in strength of the ECB connections with respect to probability of collapse of the building.

3. Design and Description of Archetype Frames

For the purposes of this study, a four story building with ECB connection was selected, considering that it is representative of a low or mid-rise building. The strength of the ECB connection was the main parameter varied. Five connection strength levels were investigated. They include the two extreme cases, i.e. fixed and pinned conditions, and three connections whose response is characterized in terms of a hysteretic model and which vary their strengths from one, representing a connection designed to the capacity of the attached column, to 0.5 and 0.3 times the former. Figure 1 schematically illustrates the SMRF examined in this paper.

Referring to Figure 1, the SMRF has four bays, resist all the seismic design loads, and receive a tributary gravity loads as indicated in the shaded portion of the plan view. The building has plan dimensions of 120ft by 180ft, respectively. The SMRF are located in the perimeter of the building on the short side (this being the analyzed direction). The bay width is 30ft, and the height of the stories is 13ft. A uniform load of 83psf is applied over each floor. An unreduced live load of 50psf is applied on all floors, whereas 20psf is applied on the roof. All beam to column connections are designed as reduced beam sections (RBS) connections in accordance with the Seismic Provisions (AISC, 2010) [11]. In terms of design, this affects column sizing due to strong column – weak girder requirement. Table 1 summarizes the design results.

Table 1: Section Sizes

Floor	Exterior Columns	Interior Columns	Beams
1	W14x342	W14x426	W30x148
2	W14x342	W14x426	W30x148
3	W14x257	W14x342	W30x148
4	W14x257	W14x342	W27x94

In order to have a comprehensive understanding of how the seismic performance of the structure would vary with respect to the change in base strength, five strength levels were defined for the analyses. The first level, i.e. fixed connection, corresponds to the assumption taken in the current design practice when modelling the connections as fixities. The inclusion of this class is important as it will enlighten the difference that exist between this widely used assumption and the real behavior of buildings. Then, three levels characterized by a phenomenological rotational spring with pinching deterioration characteristics are included. The first of this, level S1, is characterised by the stiffness and yield moment calculated as per IBC SEAOC Structural/Seismic Design Manual [13]. This is a representation of a connection designed according to the current practice specifications. This means that the yield moment is calculated to the capacity of the attached column. The subsequent three levels are expressed in terms of a yield moment ratio with respect to S1. This levels are therefore denominated S0.5 and S0.3, with the factor representing the strength of this connections with respect to the designed to current codes. This classes will capture the real behavior of the column bases as designed and also the effect of reducing their strength in order to set them as yielding elements and take advantage of their excellent dissipative properties. Note that in all this four levels the stiffness is kept constant to the one calculated to S1. Finally, a last class corresponding to a pinned connection is defined. The different values for the elastic rotational stiffness and yield capacity of all the six connection levels are summarized in the Table 2. These stiffness assumptions imply distinctive characteristic not only in the building response but also in its dynamic properties, specifically in terms of its ductility and natural period. Table 3 summarizes the properties of the building corresponding at each strength level.

Table 2: Connection Strength Levels

Properties	**Fix** $\rho = \infty$	**S1** $\rho = 1$	**S0.5** $\rho = 0.5$	**S0.3** $\rho = 0.3$	**Pin** $\rho \approx 0$
K_{ext} [kip-in]x10^6	∞	1.38	1.38	1.38	0.0009
K_{int} [kip-in]x10^6	∞	2.76	2.76	2.76	0.0009
$M_{y\ ext}$ [kip-in]x10^4	∞	44.72	22.36	13.42	0.027
$M_{y\ int}$ [kip-in]x10^4	∞	57.83	28.92	17.35	

Table 3: Natural Periods and Spectral Acceleration

Base Connection Type	**First Mode Period T_1 [sec]**	**Spectral Acceleration $S_a(T_1,5\%)$[g]**
Fixed	0.93	0.76
Strength level 1	1.05	0.65
Strength level 0.5	1.05	0.65
Strength level 0.3	1.05	0.65
Pinned	1.28	0.55

4. Nonlinear Simulation of the Building

The frame is modeled as plane frame. The column bases are modeled with rotational spring with pinching behavior with varying degrees of stiffness [16]. Table 2 introduced in the previous section summarizes the different levels of stiffness considered. The rest of the frame is modeled to effectively capture geometric and material nonlinearity. The software program OpenSEES [17] is used for all simulations. The beams and columns were implemented by means of non-linear beam column (force based) elements. The beam and column sections were included on the OpenSEES model using Fiber Sections [18]. Fiber Sections are sub-divided in patched, which constitute a sub-region of a part of the section with regular shape and that contains a uniaxial material which sets Bernoulli beam considerations. For this model and indicated by Shaw (2010) [18], each beam and column sections were modelled by dividing them into three areas, the two flanges and the web, and assigning to them a patch configuration of 16 by 4. Other assumed modeling conditions included were the enforcement of a Rigid Diaphragm on each floor, the inclusion of Reduced Beam Sections (RBS) and the modelling of rigid link zones in the beam to column connections. The Rigid Diaphragm was imposed by fixing the lateral displacements of all nodes on the same floor to be the same. The inclusion of the RBS, close to the beam column connections, ensured that the location in which plastic hinge formed on the beams was controlled, thus complying with the real design characteristics of the building (Shaw 2010). The beam and column sections in the intersection joints were modelled as Rigid Beams to account for the fact that those zones have an increased resistance. Two main types of analyses are conducted in this study: 1) Nonlinear Static Pushover analysis (NSP) and 2) Incremental Dynamic Analysis. These methods are briefly described.

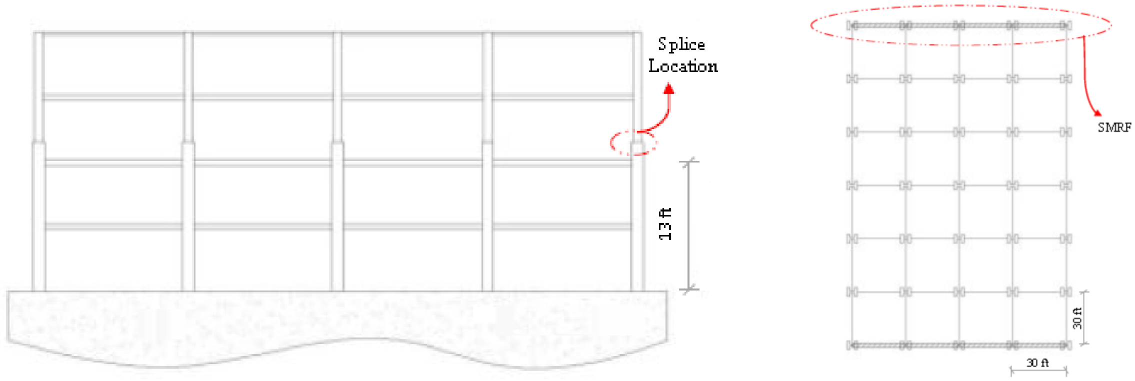

Figure 1: Plan and Elevation Sketch of Archetype Frame, modified from [18]

Nonlinear Static Pushover Analysis involves applying a predetermined pattern of lateral loads over the height of the structure. The loads are applied in a progressively increasing manner as the structure gains (or losses) strength. The results are the so called "pushover curves" wherein the deformation of a node control is plotted against the lateral base shear. The NSP analysis is performed as per Section 6.3 of FEMA P695 (FEMA, 2009). The NSP analysis has the following characteristics: (1) The gravity loads were applied through a plain load pattern and held constant in the model as they act permanently on the structure; (2) NSP analysis was performed using a displacement controlled analysis to a maximum drift of 15% in increments of 0.0001 times the height of the building and (3) a triangular shaped force distribution with an intensity of unity was applied as lateral force.

The Nonlinear Response History (NRH) simulations are conducted to evaluate the effect of base strength on the collapse potential of the building as well as the performance at defined level shaking. For the purpose of this study, the performance objectives were selected in terms of drift levels. This is reasonable given that buildings of this kind would be drift controlled rather than force controlled in terms of design. As exposed by Ghobarah (2001) [19], it is acknowledged that the specific damage states associated to drift levels may vary depending on the structure. The drift of a building is affected by several factors. They include the stiffness, the strength and the ductility of the structure. The current practice in PBEE sets some general drift levels that can be safely adopted and that provide meaningful estimates to improve the safety and economic aspects of the structures. These drift thresholds are associated to performance objectives, which as defined by Ghobarah (2001) [19], are statements of acceptable performance of the building. Table 4 exposes the drift levels selected in this study.

The last two performance objectives, the associated with 5% and 10% drift, where selected by the authors to track the response on extreme events and to try to characterize collapse. Normally, in an analysis such as the MSA, collapse would be defined by conditions like instability on the building response or reduction of the elastic stiffness in 80%. However, when this conditions cannot be met, due to the characteristics of the analysis, a drift level is selected as the capacity of the structure. In this case, the latter described conditions would not be met as global deterioration of the model is disregarded. Thus, and following the suggestions of Lee and Foutch (2002), a drift of 10% is chosen to indicate global and complete collapse of the 4 story building.

The ground motion records are part of the ATC-63 project (FEMA 2009) [10]. In total, 32 far-field acceleration histories were used. They have the following general characteristics: (1) The magnitude of the earthquake records is larger than 6.5; (2) The distance to source for the far-field set of the ATC-63 is of more than 10 km; (3) the ground motions correspond to strike slip and thrust fault mechanisms; (4) The specified soil conditions for the set are of stiff or rock soil, with a Vs > 180 m/s; (5) The recording site is specified as a free-field or ground floor of a small building; and finally, (6) The Peak Ground Acceleration (PGA) is greater than 0.2g, the Peak Ground Velocity (PGV) greater than 15 cm/s and the Lowest Unstable Frequency (LUF) of the record is lower than 0.25 Hz. Thus, the use of this wide array of records provides the possibility of studying the response of the structure to distinct events and hereby to make relatively more certain predictions about its performance.

Table 4: Performance Levels

Performance Level	Damage State	Drift (%)
Immediate Occupancy	No Damage	0.2
Operational	Repairable	0.5
Life Safe	Irreparable	1.5
Near Collapse	Severe	2.5
Near Collapse	Severe	5
Collapse	Collapse	10

5. Discussion of Results

Figure 2, represents the variation in the level of drift at the design spectral acceleration with respect to change in strength level. In all the figures (Figures 2 to 14), the fixed and pinned assumptions have been represented as boundaries rather than as discrete points so as to facilitate the comparison of the results. It is evident from Figure 2 that the level of drift experienced by the structure at the design S_a varies increasingly with decreasing connection strength as expected. The maximum value that is specified in design for drifts at the design S_a is around 2.0% to 2.5%. The figure shows that all connection levels compliment this requirement. The maximum variation between the connections with the hysteretic spring is of around 0.23 % (from 1.65% in S1 to 1.88% in S0.3). The middle strengths (S0.8 and S0.5), show little variation between them. The difference between the assumptions and these four classes is notable. The fix assumption results in drifts as much as 0.93%, a value that underestimates the real response of the structure. This can eventually have important implications in terms of serviceability and even in safety, as the assumed response does not produce the real level of displacements for the design level accelerations. On the other hand, the pinned assumption overestimates the displacements, thus giving estimates on the safe side. It is important note is that characterizing the bases as yielding elements is advantageous in terms of the drift predictions for the design S_a, as it better captures the real response and still gives results that ensure the adequate performance of the building.

Then, Figures 3 to 8 are concerned with the median variation of the S_a at the different performance levels with respect to the change in base class. At all drift levels, the fixed assumption, especially at low/serviceability levels of drift, overestimates the median S_a needed to cause that level of drift. In contrast, the pinned bases are always conservative. It has to be highlighted that at the 10% drift level, any of the ground motions reached this “collapse” state for the fixed assumption, therefore giving more credit to the fact that modelling connections in this fashion can lead to erroneous estimates of the performance of the building.

For low levels of drift, i.e. for 0.2 and 0.5%, the connections with the hysteretic model exhibit no variation in the median S_a. Also, it can be seen that they are more closely represented by the pinned assumption. Their similarity lies in the fact that this levels represent the Immediate Occupancy and Operational states of the structure. Thus, the proposed approach would entail that the bases would not yield at this levels and thus their response is the same. For the intermediate levels of drift 1.5%, 2.5% and 5%, which correspond to life safe and near collapse states respectively, the structural response has a common trend with respect to the variation in strength class. Clearly, as the drift level increases the difference between these connection levels increases. Initially, at level 1.5%, only the S0.3 connection shows a decrease in median S_a. Then, the number of classes decreasing their median with respect to S1 progressively increases with increasing the drift level examined. Eventually, at 5% drift all the different strength levels show a clear decreasing trend in median spectral acceleration. Here, the effects of the variation in strength, i.e. in yield moment as defined in the hysteretic model, are clearly captured.

As the intensity of the ground motions increase, and so the drift does, the different connection classes start to yield in ascending order of strength, thus producing the progressively descending pattern observed in the results. An important point to drawn from these figures is that this trend only starts to be notable for all strengths after the life safe level (1.5%), where all the connections but S0.3 respond in the same manner. This suggest that for this performance objectives, the level of strength of the connection only would become critical at very low values with respect to the designed as per current codes (S1). As pointed out this differences increase and become more notable and thus crucial at larger drift states (near collapse).

Previous decreasing trend only changes at the 10% drift level. At this level, the connections with lower strengths perform better than the designed as per code. This could be explained by the fact that this lowering in strength can enhance the ductility of the connections and thus, in limit states such as collapse they can produce a better performance with respect to more robust bases. This fashion also suggests that if the level of drift was increased further, by defining collapse at larger values, the performance of the connections will again stabilize. It can be indicated that for connections designed as yielding elements and with 20% lower strength that of the attached columns perform almost identically to the ones designed as S1 in terms of drift. Also, the S1 and S0.8 connections showed reasonable S_a measures for all the drift levels, ranging from 0.05g to about 2.9g for the lightest and extreme cases relatively.

Figures 9 to 14 represent the change in median rotation with respect to the different strength connection levels for a certain drift performance measure. The fixed assumption connection produced zero rotations at all levels of drift as its definition in terms of modelling implies. When appropriate, a 50 mmrads tag is plotted, to indicate a maximum allowed rotation defined in reference to codes of practice. Again, the pinned assumption shows conservative results for all levels of drift. Also, a similar trend, associated with the yielding of the connections, can be observed with the increasing levels of drift. For 0.2% and 0.5% all the rotations remain between 5 and 6 mmrads for all connection strengths. At 1.5% drift, all connections kept this trend, having constant values between 10 and 15 mmrads. However, there was a particular lowering in in the rotation for the S0.3 level. For higher drift levels the rotations subsequently increase. At 5% drift, the second near collapse state, the rotations for the lower connection levels surpass the threshold of 50 mmrads, indicating excessive demands in the bases. At 10% drift, all the connection levels are well over the 50 mmrads with rotations around the 100 mmrads. However they keep the same increasing trend with decreasing strengths. Rotations thus characterized another critical aspect to attend when assessing the performance of the structure. Although the response generally maps that described by the S_a levels at the different drifts, at limit states rotations become more critical as they become very large.

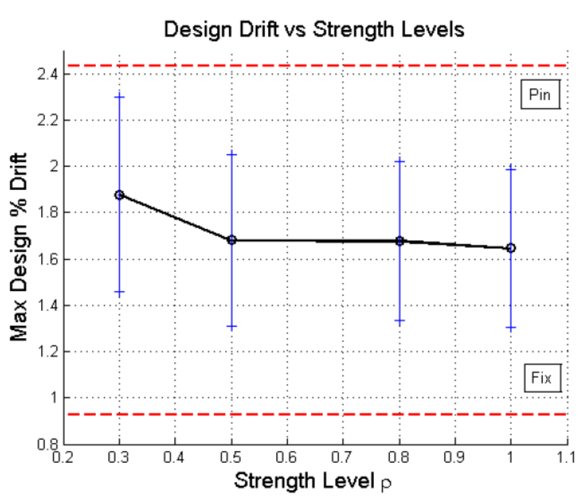

Figure 2: Drift at Design Spectral Acceleration

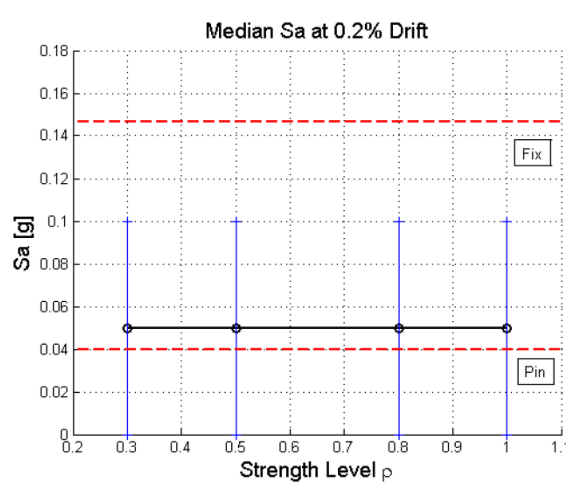

Figure 3: Spectral Acc. at 0.2% Drift Level

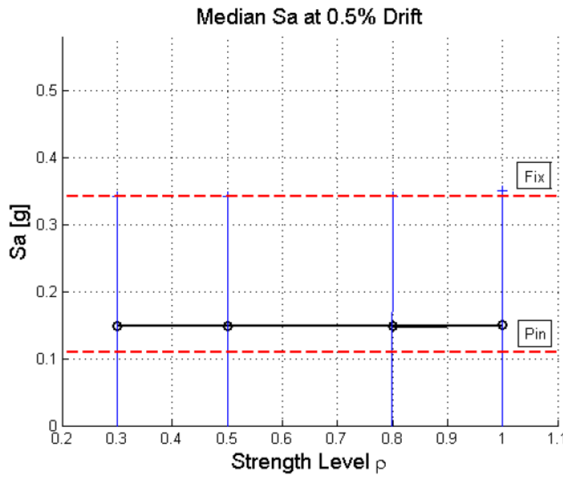

Figure 4: Spectral Acc. at 0.5% Drift Level

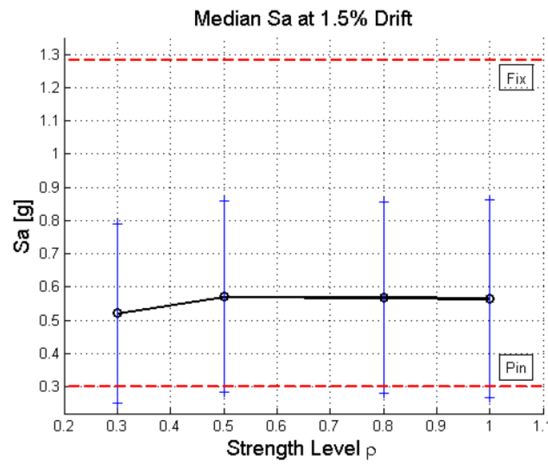

Figure 5: Spectral Acc. at 1.5% Drift Level

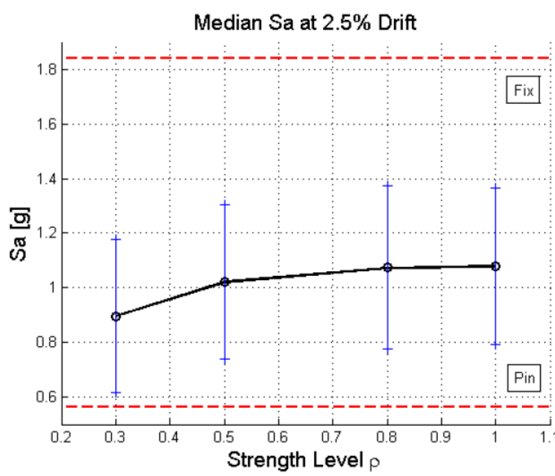

Figure 6: Spectral Acc. at 2.5% Drift Level

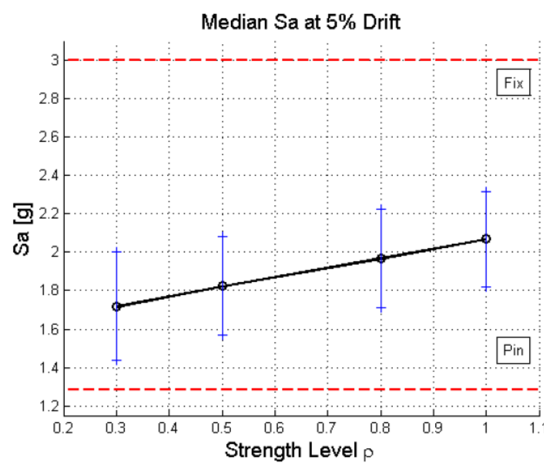

Figure 7: Spectral Acc. at 5% Drift Level

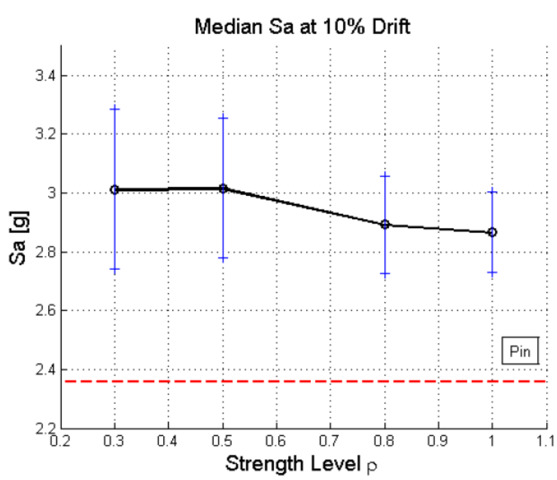

Figure 8: Spectral Acc. at 10% Drift Level

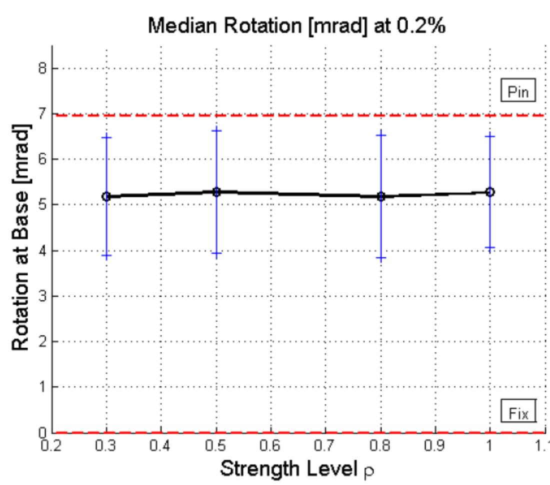

Figure 9: Rotation at 0.2% Drift Level

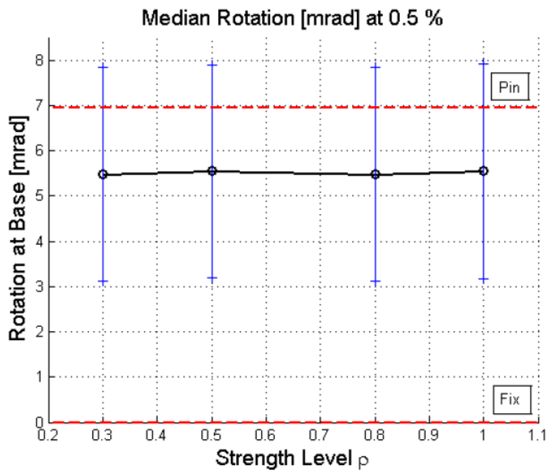

Figure 10: Rotation at 0.5% Drift Level

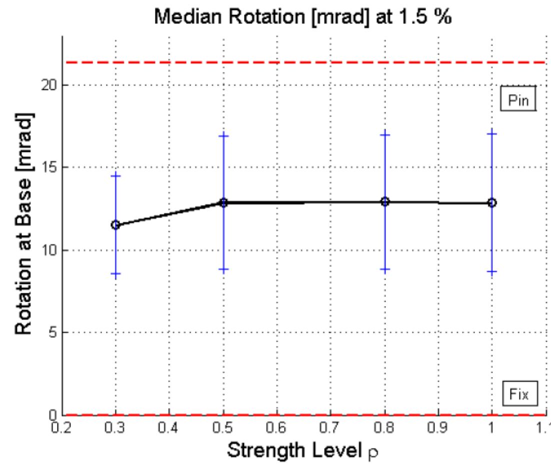

Figure 11: Rotation at 1.5% Drift Level

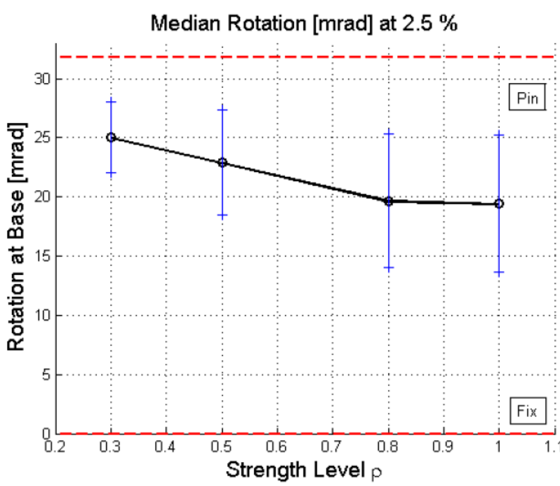

Figure 12: Rotation at 2.5% Drift Level

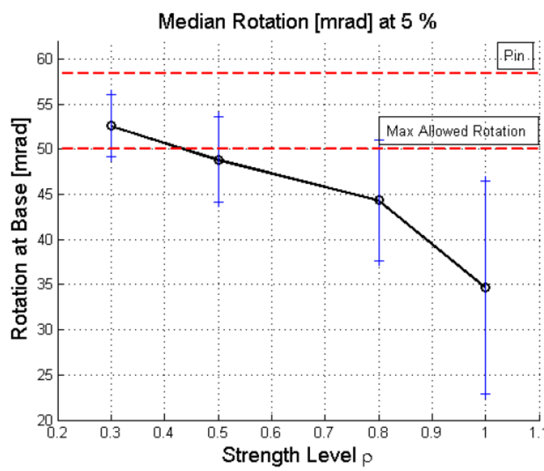

Figure 13: Rotation at 5% Drift Level

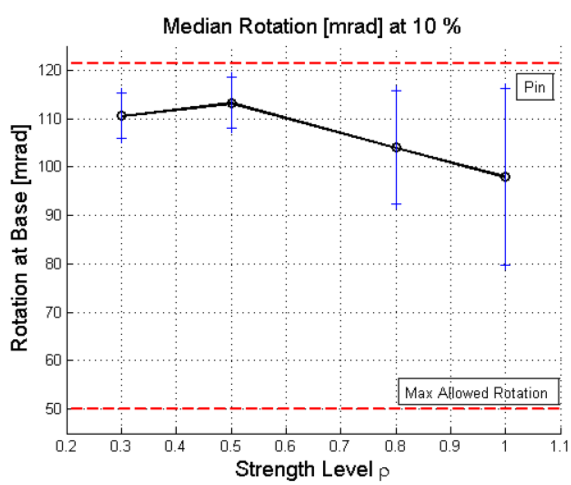

Figure 14: Rotation at 10% Drift Level

6. Acknowledgements

The authors would like to recognize Dr. Christian Malaga-Chuquitaype, from Imperial College London, who gave support and advice to the authors during the early stages of this investigation. The authors would also like to thank The National Secretariat of Higher Education, Science, Technology, and Innovation of Ecuador (SENESCYT) whose graduate fellowship provided support for the second. author.

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