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The seismic performance of structural steel buildings in the 2010/2011 Christchurch earthquake series and the 2016 Kaikoura earthquake: lessons learned, ongoing research and needs of industry

K.T. Mohandas & S. Ramhormozian

Department of Built Environment Engineering, Auckland University of Technology, Auckland.

G.C. Clifton

Department of Civil and Environmental Engineering, University of Auckland, Auckland.

G.A. MacRae

Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch.

ABSTRACT

At least 20 earthquakes greater than Ms 7 have taken place in New Zealand since 1850. The recent earthquake that occurred in Kaikoura (Ms 7.8) is considered one of the largest earthquakes since 1855, but Christchurch's occurrence in February 2011 was one of the most severe and devastating incidents with fatalities. Although there were examples of damage that occurred to steel buildings and structures, the overall performance of the steel buildings and the steel structures were good. This research paper intends to analyze the performance of structural steel from the Christchurch and the Kaikoura earthquakes, lessons learned, current research in structural steel, needs of industry, and current structural steel market share in New Zealand. Structural steel has risen up from around 0% market share to almost 80% in Christchurch after the earthquake series, mainly due to its performance. However, research is being continued on various seismic load resisting systems to improve its performance further. Concerns regarding structural steel bolting and recommendations are also described succinctly in this research paper.

1 CHRISTCHURCH AND KAIKOURA EARTHQUAKES

Christchurch was struck with a series of six destructive seismic occurrences during the period of 2010-2011. The seismic occurrence on 4 September 2010 (Ms7.1 & MM7) and on 22 February 2011 (Ms 6.3 & MM9-10 in CBD) were the most serious of these series (Clifton et al., 2011). Although much damage occurred due to these, no fatalities were recorded in the 4th September incident. However, the 22nd February occurrence was more destructive and life taking incident. In the latter incident, the epicenter was closer to the Christchurch CBD and the hypocentre was 5km below the ground (Clifton et al., 2011), whereas it was 10km deep in case of September event.

Another major earthquake occurred in New Zealand in Kaikoura on November 14, 2016. The magnitude was 7.8 and is considered the largest seismic occurrence since the 1855 earthquake, but no state of emergency was declared unlikely to 2011 Christchurch incident (HERA, 2017). This is due to limited damage and to avert any major financial disturbance such as occurred in Christchurch following the long evacuation of the CBD (HERA, 2017).

2 RESEARCH OBJECTIVE

A series of catastrophic earthquakes occurred in Christchurch and Kaikoura in this decade and have caused a wide range of damage. This paper presents analysis of the structural steel performance during these seismic occurrences, lessons learned, current research in seismic damage-resistant design, industry needs, and structural steel market share in New Zealand.

3 LITERATURE REVIEW

3.1 Background of Steel Buildings in New Zealand

In the early 20th century to 1960s, riveted steel frames were a typical design (HERA (2017)). However, the riveting was changed to welding and bolting designs by the 1960s and was commonly used in commercial and residential structures. As a result of industrial relations problems, the use of structural steel had been banned in most of the construction projects by the mid of the 1970s. The market share of steel dropped to about 0% by starting of the 1980s. A minor and gradual rise in structural steel embarked again from 1990 in the construction industry in New Zealand.

The market share of structural steel in Christchurch was very low up to 2000 but had increased to be around 50% by floor area supported prior to the earthquake series in 2010 – 2011. This was mainly because of the excess availability of affordable aggregates for concrete, and high focus on education and research of reinforced concrete in construction (HERA, 2017). Post-earthquake, structural steel played a critical part in the reconstruction of Christchurch, rising the structural steel market share by floor area above 80% in Christchurch. The primary reason for this is the good performance of steel structures and buildings during these severe seismic occurrences.

Different types of common seismic load resisting systems or designs are:

1. Moment resisting frames
2. Centrally braced frames
3. Eccentrically braced frames

The use of the above systems in the steel buildings, while being well designed and detailed, is one of the primary reasons for its good performance during earthquakes. Research is being carried out on these design systems for improved seismic performance known as low damage seismic resisting systems.

The seismic resisting steel buildings in New Zealand are designed using moment resisting frames (MRF), concentrically braced frames (CBF), eccentrically braced frames (EBF), or by a combination of MRF and EBF. Even though structures or buildings are allowed to design with high ductility, many buildings are designed with low ductility, or even elastic response (G. MacRae, Clifton, & Innovations, 2013). The reason for this is that member sizes of a structure are often controlled or determined by not only earthquake loads but also loads like gravity load, wind load, seismic inter storey drift, etc. Member overstrength due to slab effects and other non-structural components can increase the strength, maybe to a limit that buildings almost respond elastically (G. MacRae et al., 2013). Overstrength may even help a building to resist severe earthquakes. This can be considered one of the major reasons for the good performance of steel building in the 2010/2011 Christchurch earthquake series. However, there were some isolated and minor problems in some buildings. Therefore, in order to be more damage resistant, research on various low damage seismic resisting systems is being continued, albeit slowly due to lack of available funding.

3.2 Attributes Impacting Seismic performance of Steel Buildings

Due to the ductile properties of the steel material, it is capable of dissipating a considerable amount of energy when subjected to inelastic forces. This ductile response of the steel members makes steel buildings suitable for even high earthquake-prone areas. However, there are various attributes that affect the steel members performance, these are the following:

- Imperfection in steel members
- Building Deterioration over time
- Load paths through connections
- Contribution from the slab
- Age of Building
- Steel Member Restraints
- P-Delta Effects

3.3 Low Damage Seismic Design of Steel Structures

In order to limit the significant damage to a building during a major earthquake, either the building can be designed to very high strength which is not economical or it can be designed to normal strength using damage resisting systems or designs and economically justifiable. (Buchanan et al., 2011)

Low damage systems are designed to minimize and, in some cases, limit the damage to certain specified components that may be easily repairable or replaceable, based on the capacity design philosophy. However, different types of systems are there, each having their own characteristics, merits, and demerits. The primary purposes of these types of systems are to control and reduce damage, and to minimize the downtime of the building or structure, thus making it reusable soon after a major earthquake. (G. A. MacRae & Clifton, 2015)

Various types of low damage seismic resisting systems are:

- Low damage systems that can be Implemented in Moment Resisting Frames
 - Post-tensioned or spring-loaded beams (G. MacRae et al., 2013)
 - Sliding Hinge Joint with asymmetric or symmetric friction connections (G. MacRae et al., 2013)
- Low Damage systems that can be implemented in Concentrically Braced Structures
 - Buckling Restrained Braces (G. MacRae et al., 2013)
 - Asymmetric Friction Connection in CBF (Leslie, Gledhill, & Moghaddassi, 2013)
- Low Damage systems that can be implemented in Eccentrically Braced Structures

- Eccentric Bracing Systems with Replaceable Link (G. MacRae et al., 2013)
- Asymmetric Friction Connection Braces in Eccentric Bracing System (G. MacRae et al., 2013)
- Asymmetric Friction Connection in the link element in EBF (G. MacRae et al., 2013)
- Other Damage Resisting Technologies are:
 - Rocking structures using spring joint (G. MacRae et al., 2013)
 - Base Isolation System (Buchanan et al., 2011)
 - Structures with Supplemental Damper (Buchanan et al., 2011)

3.4 Performance of Structural Steel during the Christchurch and Kaikoura earthquakes

In the series of earthquakes that occurred in 2010-2011 in Christchurch, February 22, 2011, widespread damage occurred. This included failure of many nonreinforced masonry buildings, the collapse of some reinforced concrete structures, structural damage to many buildings, and the occurrence of intense soil liquefaction. Although the strong vibration duration of each seismic occurrence was low, the cumulative strong shaking recorded was more than 60 seconds. Over 2.5 times the ULS peak ground acceleration was recorded for the 22 February 2011 earthquake. (Clifton et al., 2011) in the Christchurch CBD, rising to 8 times at the location of the maximum PGA recorded.

From 1990s market share of structural steel in New Zealand started rising up from almost 0%, hence most of the steel structures in Christchurch were constructed in recent decades prior to the earthquake series and incorporated with seismic load resisting systems. Structural steel buildings and other steel structures like bridges and historical structures showed an overall good performance during these series of seismic incidents which went up to 2ULS.

Multi-storey buildings with EBF exhibited good performance. The Club tower, 12 storey building, had yielding of the active links on every level but none of these required replacement. There was also minor damage to non-structural components which was readily repaired. This building was the first normal importance building that was back into operation by July 2011 after the completion of the repair works. Another multi-storey building, Pacific Residential building is a 22-level tower that was identified with structural damage. One of the active links or the link elements in the EBF in level 6 was fractured (figure 1 and 2). However, this kind of damage/fracture was not expected in this case. (Clifton et al., 2011). Many other active links underwent yielding, with seven being replaced because of the magnitude of the plastic shear strain reached.



Figure 1. Fractured EBF link in Pacific Residential Building (Clifton, Bruneau, MacRae, Leon, & Fussell, 2011)



Figure 2. EBF link after removing from its location. (Ferguson et al., 2013)

The EBF link in level 6 of the Pacific building was fractured, the first time this type of damage was reported globally. This led to the examination of the fractured link. The active link of an EBF is designed to yield

before yielding of the other primary members like bracing, beam or column. In this particular case, no sign of plastic deformation was found on the upper flange and web, rather the link was fractured like a brittle material. The EBF was a K Type arrangement. Careful examination of the fracture showed that the starting point was the shear stud weld. Shear studs were welded on top of beams for embedding concrete slabs to the beams. One of the shear studs were welded on top of the link element at one corner directly above the stiffener of the link beam. Shear stud welding melts a small part of stud material and base steel and fuses them together. Metallurgical and hardness inspections on the weld area indicated that there were slag inclusions and untampered martensite. Also, the location of the shear stud on the corner of the link element was the area that should have been yielded in case of high lateral load. From the visual inspections, it was clear that the fracture started from the weld and spread through upper flange and web to the bottom flange of the link beam, where it got arrested 3 times probably in 3 seismic events. There is a chance that a large crack already pre-existed prior to the first seismic occurrence in September 2010 due to corrosion and concrete on the fracture at the weld area. Scanning fractography indicated that the entire fracture was dominated by a cleavage. From this analysis, it was concluded that the isolated case of the active link failure in level 6 of the pacific tower was due to (1) the particular location of the shear stud at the actual yielding area of the link element (2) the low impact fracture energy of material at surrounding temperature (3) the impact loading due to seismic occurrences. (Ferguson et al., 2013)

Clifton et al. (2011) A three-level car parking of a shopping mall with the EBF system performed well. There was no trace of plastic deformation found on the link element. These frames were designed to take 3 more levels of car parking, and the seismic vibration intensity in this area was lesser than the Christchurch CBD shaking, these can be considered as additional factors for its outstanding performance.

An EBF in a hospital car parking also performed reasonably well. However, fracture in link elements was found in two braced bays. The fractures indicated a ductile overload failure. Closer examination of the damage analysed that the reason for this damage is primarily because the flange of the bracing is not in line with the stiffeners of the link beam and is welded at an offset from the stiffeners (figure 3). This caused severe overload between the junction of beam flange and beam web, resulting in a fracture in the junction and the fracture was spread to the beam web. From this fracture or damage, it can be concluded that the load path through the as-built detail is very crucial. This was a design issue; the joint was fabricated in accordance with the engineer's drawings and shows the importance of getting the load paths correct through these connections



Figure 3. Fracture in EBF Link due to inadequate load-path (Clifton et al., 2011)

Clifton et al. (2011) stated in his research that many seismic loads resisting systems are hidden by non-structural or architectural components which is a hindrance for inspection of the structural elements. In these

cases, whether damage occurred in the structural element can be inferred only if the non-structural elements on top of it are damaged. Due to this, he suggested that future building code committees may consider designing the buildings in such a way to provide easy access to the inspection of various critical structural components as well as non-structural components.

A car parking with CBF systems exhibited partly poor performance. There were 2 braced bays, one bay on each side of the parking garage door. The one on the west side, columns were connected with a steel beam at the top, whereas in the east side bay was not similarly aligned with the beam. East side bracing bay performed badly, the weld joint between the bracing and column was fractured due to tension loads at the welds. The primary cause was that the weld was not designed according to the capacity design principles of NZ 3404 and couldn't develop the tension capacity of the brace. Whereas the westside braced bay performed better with no kind of fractures, but residual buckling was found as a result of bracing elongation. (Clifton et al., 2011)

All the inspected multi-level buildings with the MRF system performed well with no damage to steel frames or structural damage. However, in a 7-storey building the perimeter frame sunk a noticeable amount with respect to the core resulting in inter storey drift which damaged the stairs and other non-structural components. (Clifton et al., 2011)

Some RC old buildings were retrofitted with steel frames that performed well in 2010 occurrence but some didn't do well in 2011 occurrences. Many warehouses performed well and some were designed with light roofs with sag rods. After the earthquake occurrences, many rods had to be retightened that were stretched due to yielding. (Clifton et al., 2011)

During the Christchurch earthquake series, the overall behaviour of structural steel was very good. Many structures were occupiable after the earthquakes. Many structures continue to be used even though there was some damage to non-structural elements. (G. MacRae et al., 2013). However, there was some isolated damage like active link element fracture in EBF or, bracing weld fracture in CBF. All these examples of damage were a result of inadequate load paths, or improper weld designing not according to the capacity design principles. These outcomes show that modern well designed and built steel structures may be described as damage resistant.

Post Christchurch seismic series, an enormous amount of rectifications and reconstructions were carried out. While most of the buildings were constructed with RC before the earthquakes, the reconstruction works included an enormous quantity of steel (Bruneau & MacRae, 2017). Hence, steel made a significant part in reconstruction and dominated over other systems. This change may be due to various factors like the seismic performance of steel, easy repairability, swiftness in steel erection or installation, economical fire protection methods, easy retrofitting ability, easy installation of new low damage technologies, reduced price of steel, etc (Bruneau & MacRae, 2017). A study was done by (Bruneau & MacRae, 2017) on 74 buildings that were constructed post-Christchurch seismic series showed that the ratio of buildings in steel, reinforced concrete, and timber lateral force resisting system was 10:10:1 respectively. However, the floor-area ratio of steel, RC, and timber load resisting systems in the same buildings are 79:20:1 respectively. Different types of lateral load resisting systems were used in the newly constructed steel buildings. The frequently used are BRB frames, MRFs, MRFs with reduced beam sections, MRFs with friction connections, EBF, EBF with the replaceable link, CBF, Rocking steel frames, base isolation buildings with MRF or CBFs.

A severe earthquake in Kaikoura in 2016, affected Wellington region. The performance of structural steel was similar to the Christchurch, it exhibited good performance. However, some damage were noticed (HERA, 2017). Wellington had 7 notable multi-level buildings with steel in which 3 were reported with observed damage. One is a three-story building with an EBF system in which the gusset plate connections between the brace and the collector beam buckled out of the plane (figure 4). The collector beam is the beam

in which the active link is located. This buckling showed the gusset plates' vulnerability. This kind of damage was not found in Christchurch buildings as the EBF connections between brace and beam were different, the braces were directly welded under the collector beam. Hence when designing connections, the out of plane actions must be properly accounted for. The second building was an industrial building that had only tension rod bracings. The tensions rods were coupled using a coupler and the damage observed was that, the failure of the bracing rod due to unscrewing of bracing from the coupler (figure 5). Since the structure had sufficient strength the building remained stable even though the tension rod bracings failed. Similar kind of issues was found in Christchurch also, loosening of sag rods. This issue might be prevented by using the lock nuts in couplers in tension rod bracing. The third was a seven storey office building. It is an RC building; the only structural steel component is the coupling beam cast into concrete. This beam works as a dissipator and designed to be the primary yielding element. This acts like the link element in the EBF system. The coupler beam exhibited some yielding and was replaced. Some of the buildings were retrofitted with seismic load resisting systems, like McKenzie building was reinforced by retrofitting Eccentrically braced frame in 2010 which could be one of the reasons for its good performance as these reinforcements would have helped to resist seismic forces.

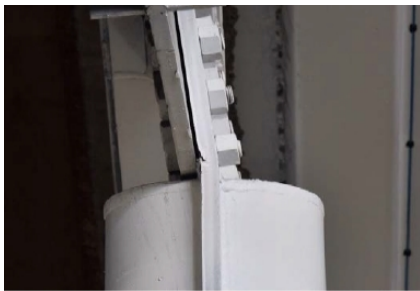


Figure 4. Gusset plate buckled out of plane in bracing. (HERA, 2017)



Figure 5. Bracing rods unscrewed from coupler (HERA, 2017)

4 LESSONS LEARNED FROM THE SEISMIC PERFORMANCE

Structural steel performed very well during the Christchurch and Kaikoura earthquakes. The market share of structural steel rose from almost 0% to around 80% in Christchurch, primarily because of the overall good performance of structural steel.

From various literature reviews, it was found that the damage resisting systems performed well as they were designed to do in both the earthquakes (Christchurch and Kaikoura). Some structural damage was found post-Christchurch earthquakes, which were not expected to occur, like active link fracture in the EBF system, weld fracture in a CBF system. Investigation of this damage led to the conclusions that these were primarily because of the construction issues, inadequate as-construction load path, and improper weld designing and not according to the capacity design principles respectively. Some structural steel damage was found after the Kaikoura earthquake also, these were out of plane buckling of gusset connection between brace and collector beam in the EBF system, the tension only brace rods unscrewed from the coupler. Investigation conclusions led to better design on gusset connections taking into consideration the out of plane buckling and use of lock nuts on tension rod brace and couplers.

5 BOLTING CONCERNS ON STRUCTURAL STEEL ERECTION

One of the most common methods of joining structural steel is bolting. This method is followed widely in structural steel buildings and other steel structures. Thus, the bolting has a very crucial part in the stability of the steel buildings or structures.

The minimum bolt tension to be attained at installation is specified in NZS3404 (1997/2001/2007) which is nearly equivalent to the minimum proof load given in AS/NZS 1252 (1996). As per NZS 3404, this can be achieved either by the part-turn method of bolt tightening or by using a direct-tension indication device. (S Ramhormozian, Clifton, Macrae, & Khoo, 2015).

The other two bolting requirements specified by NZS 3404 are:

- The projection of bolt from the nut face shall be one clear thread after tightening. This is to ensure that full thread is engaged over the total depth of nut.
- The minimum number of clear threads run out beneath the nut after tightening for fully tensioned HSFG bolts. This required the existence of five, seven, or ten free threads under the nut loaded face with respect to different bolt length to diameter ratios. This is for avoiding excessive plastic strain demand in the thread area of the grip length when the bolt is fully tensioned.

(S Ramhormozian, Clifton, Macrae, et al., 2015) conducted bolt tightening tests on HSFG class 8.8 bolts procured from two different New Zealand based suppliers, and the actual bolt tension after the installation was found to be very much below the specified values in the code. A few bolts also fractured during the tightening test, at a tension well below the yielding point i.e. 50% of the proof load. The failure/fracture of bolts led (S Ramhormozian, Clifton, Macrae, et al., 2015) to carry out tensile testing on the bolt samples to analyse and confirm the mechanical properties of the bolts.

The reasons for this non-compliance were investigated and following were the observations by (S Ramhormozian, Clifton, Macrae, et al., 2015):

- The materials properties of the bolts were above the specified by the bolting standard by tensile tests.
- The bolts with no visible lubrication as well as with rough galvanized surface showed more resistance against turning the nut and poor behaviour during tightening.
- All bolts that were tightened to the elastic tension range showed permanent elongation after untightening.
- The torque applied as well as nut turn for a given bolt tension were higher than expected.
- The tension vs length curve during tightening was found to be non-linear even in the elastic range.
- Tensile tests showed necking resulting in a ductile behaviour but different in appearance, suggesting a different mode of fracture.

From the above observations, it was concluded by (S Ramhormozian, Clifton, Macrae, et al., 2015) that the poor thread and finish quality of bolts were the major reasons for the bolts not meeting the code specified tension values as well as the fracture.

Based on the above-observed concerns, the following recommendations were made by (S Ramhormozian, Clifton, Macrae, et al., 2015):

- Pre-installation of the nut on the bolt by turning it to full length by hand and ensure that the nut turns freely and smoothly the full thread length of the bolt.
- Even though the clause 3.2.5.4 of AS/NZS 1252 specifies that the bolts should be supplied lubricated, it is recommended by (S Ramhormozian, Clifton, Macrae, et al., 2015) that the bolts shall be applied with appropriate lubricant before installation. E.g. molybdenum disulphide paste.
- Do not reuse the previously fully tensioned bolts.
- The research reconfirms the NZS 3404 requirement that the bolts are not to be fully tensioned via torque setting.
- Turning bolt head during tightening can be a potential sign of over-torquing and risk of plastic torque development.

- The required part-turn shall be identified based on the grip and thread length rather than the bolt length. This is because the grip and thread length of a bolt can give the longitudinal stiffness of the bolt that be correlated to the required turn of the nut.

(S Ramhormozian, Clifton, Nguyen, & Cowle, 2015) conducted another set of bolt tightening tests to investigate the practical and theoretical concerns on the NZS3404 recommended part-turn method of HSFG bolt tightening. In practice, it was sometimes difficult for the industries to meet the recommended number of free threaded on the nut loaded face, hence this research was conducted by (S Ramhormozian, Clifton, Nguyen, et al., 2015) to understand if this could be relaxed. The theoretical concern is that to reach the required bolt tension, the applied part-turn creates a longitudinal strain beyond the yield on the loaded part of the bolt. The unloaded side of the bolt is the only part of the bolt which does not accommodate this applied strain. Thus, the unloaded part of the bolt should be excluded to determine the required part-turn or this could lead to over tightening of bolts with short grip lengths.

(S Ramhormozian, Clifton, Nguyen, et al., 2015) followed the recommendations suggested by (S Ramhormozian, Clifton, Macrae, et al., 2015) on bolts and performed tensile tests on the M20 X 100 with 50mm shank and M24 X 110 with 55mm shank bolt samples to ensure it met the material and mechanical properties specified in the manufacturer certificates. They used plates of different thickness as plies in order to achieve the different grip lengths for the tightening test. The bolt tightening test was performed and table 1 below shows the applied turn to achieve the minimum specified tension value in the code after snug tightening for the different number of free threads on the nut loaded face.

Table 1: Applied Part-turn for different number of free threads (S Ramhormozian, Clifton, Nguyen, et al., 2015)

The number of free threads at nut loaded face	The applied part-turn			
	M20 Bolt		M24 Bolt	
	Turn	Degree	Turn	Degree
1	1/4	90	1/4	90
3	1/4	90	1/4	90
4	1/4	90	1/4	90
5	1/4	90	1/4	90
6	3/8	135	1/4	90
7	1/2	180	1/2	180
8	1/2	180	1/2	180

Shahab Ramhormozian, Clifton, and Cowie (2016) concluded that the nut rotation in part turn method is primarily related to and governed by the ply & washer thickness and not by the bolt or shank to thread lengths ratio.

Shahab Ramhormozian et al. (2016) made the following recommendations to be amended in NZS 3404:

- *“Nuts shall run freely when part of a bolt/nut assembly. This shall be checked by running the nut along the bolts threads by hand the full length of the thread before being used in a connection. Only bolt/nut assemblies that pass this test shall be used. Once a bolt/nut assembly has been so tested neither component shall be substituted”*
- *“For tensioned bolts to either 15.2.5.2 or 15.2.5.3, at least two clear threads run out shall be clear beneath the nut after tightening”*
- The bolt length is defined as the *underside of the bolt head to the outer face of the nut.*

6 QUESTIONS TO BE ANSWERED AND ANALYZED TO PLAN FOR THE FUTURE

This research was carried out by analysing the performance of structural steel during the earthquake series in Christchurch & Kaikoura and deriving the outcomes of it through the literature reviews. Various questions were formulated from the ongoing research by the authors for interviewing professional entities like the organizational body, steel fabrication company, and academic people. This shall be carried out further in the near future. The formulated questions are as follows:

1. What is the current market share of structural steel by floor area in NZ?
2. Christchurch was not the most seismic prone risk zone when the major series of earthquakes occurred in 2010/2011, delivering the highest PGA recorded worldwide to date. This shows that the actual intensity can be very much greater than that predicted by the NZ seismic risk model.
3. If the seismic HAZARD risk factor of regions is varying over a long period of time, should the buildings be retrofitted as per new demand and should all the new buildings that are being designed or constructed given the facility to retrofit in the future? Are there any valuable pieces of information on wellington retrofitting?
4. What are the ongoing research needs and current/future trends on damage resisting systems? What parameters, such as post-earthquakes' experiences, influenced this trend?
5. EBF link failure/fracture in the pacific tower due to the stud location on the yielding area: If this was not in yielding area wouldn't the result have been similar, fracturing the beam flange and web? How does a stud welded with adequate quality on top of an EBF link affects the performance of the link?
6. Are steel fabricators/manufacturers facing any difficulty in fabrication or erection of any kind of damage resisting systems like MRF, CBF, and EBF, sliding hinge joint, or any low damage steel systems? If yes, are there any suggestions to make it more fabrication or installation-friendly?
7. How aware are a Steel fabricator/manufacturer and its team on different seismic damage resisting systems? One of the reasons for structural members' failures in Christchurch earthquakes is the construction error. What could be the strategies adopted by the Fabricator in making its manufacturing unit, erection unit, and the quality team makes aware of the do and don'ts in each system?
8. Inadequate load path was a reason for the failure of some steel members in the Christchurch earthquake series. The examples seen were all design errors. This raises the issue of checking of designs and the importance of load paths. It also illustrates the susceptibility of steel connections to poor performance if the load paths through the connections are not properly determined, designed and detailed.
9. The quality of fabricated structural steel plays a major role in supporting the seismic performance of a steel building. Imperfections in fabricated steel is a factor for reducing the seismic performance of steel, how do fabricators check and overcome this. Steel Construction New Zealand have made major advances in this area with their SCNZ Accredited Fabricator scheme
10. What are the advantages or disadvantages of structural steel buildings and structures with respect to other materials (RC, Timber, Light Gauge Steel, and Masonry)?
11. If in case of retrofitting required for a building, how do you compare the modification works on a steel member and in general structure with any other material type in general?
12. How easy is it to repair a damaged steel structure or steel member when compared to any other materials?
13. Are there any key challenges in terms of structural steel design procedures that might demotivate designers to select steel as the first option?
14. How well and based on what procedure designers and erectors work together? Is there any room for improvement?

7 CONCLUSION

Although structural steel performed well during the last decade of earthquakes, research is continuing on various low damage seismic designs to further improve its performance, and there is still considerable room to address the issues and improve the systems. The presented research is focused on ongoing research and industry needs dealing with the structural steel in New Zealand.

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