Conceptual Investigation of the Northridge's Earthquake Structural Failures on Steel MRF by the Aid of the Swiss Cheese Model

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Anthimos Anastasiadis[1], Marius Mosoarca[2], Antonio Formisano

[1] ASAnastasiadis & Associate, Consultant Engineers, Thessaloniki, Greece
[2] Politechnica University of Timisoara, Department of Architecture, Timisoara, Romania
[3] University of Naples Federico II, Department of Structures for Engineering & Architecture, Naples, Italy
anastasiadis@asacon.eu

Abstract. On January 17, 1994, the Northridge earthquake, USA, struck the southern California area, causing unexpected damage to beam-to-column welded steel moment-resting frames (WMRF. After 30 years of this benchmark earthquake, it is interesting to investigate this event from a different point of view, namely, through a combined consideration of forensic engineering and risk management. The Swiss Cheese Accident Model provides an excellent framework for understanding how a series of minor or major errors or failures can align to result in a catastrophic event. The study presents an effort to approach the failure of the Northridge earthquake of steel WMRF through the view of the Swiss Cheese accidental model. The investigation identified the layers of defense and their associated holes. Moreover, we discussed the weaknesses, corrective actions, and improvement actions.

Keywords: Northridge earthquake, Steel moment resisting frames, Forensic engineering, Risk management, Swiss Cheese accidental model.

1 Introduction

On January 17, 1994, the Northridge earthquake in Los Angeles, California, USA, struck the area at 6.8 magnitude on the Richter scale, causing unexpected damage to welded steel moment-resisting frames (WMRF). A systematic failure was observed without any sign of plastic deformation of beam-to-column connections. The structural design, construction, and inspection practices, along with seismic action characteristics, were also attributed to this unexpected damage. As a result, a paradigm shift occurred with this earthquake event. In this direction and associated with the steel moment-resisting frames (MRF), new constructional conformation, details, practices, and design methodologies were introduced.

After 30 years of this benchmark earthquake, it is interesting to investigate this event from a different point of view, namely through a combined consideration of forensic engineering and risk management. The Swiss Cheese Accident Model (SCAM) provides an excellent framework for understanding how a series of minor or major errors or failures can align to result in a catastrophic event. It is more generic and prescriptive, and it is better adapted to the case under investigation than Heinrich's Domino Accident Model. Originally proposed by James Reason in 1990 [1], the Swiss Cheese Accident Model depicts an organization's defenses against failure as multiple layers of Swiss cheese. These layers represent procedures, processes, and safeguards put in place to prevent accidents. However, just like slices of Swiss cheese, each layer possesses holes or weaknesses. A trajectory for accident causation becomes available when the holes in each layer momentarily align, thus resulting in a system failure. The SCAM gained significant recognition through the safety engineering community, finding widespread application in different industries such as aviation, nuclear power, oil and gas, healthcare, and the construction industry [2, 3].

The study presents an effort to approach the failure of the Northridge earthquake of steel WMRF through the view of the Swiss Cheese accidental model. This research adopted a qualitative case study approach. The investigation identified the layers of defense that contributed to the failure (accident) and their associated holes. It attempts to report the main factors that contributed to the aforementioned failure; further on, it discusses the weakness and the corrective and improvement actions (even those that should be performed) for the betterment of the steel construction industry.

2 From San Francisco, 1906, to Northridge, 1994 and post Northridge earthquake era

Three distinct periods must be considered with regard to the analysis and investigation of the unexpected failure of welded MRF connections. The first one starts from the San Francisco earthquake in 1906 until the Northridge earthquake in 1994 (passing through the Santa Barbara, 1925, Long Beach, 1933, Imperial Valley, 1940, 1979, Alaska, 1964, San Fernando, 1971, Whittier Narrow, 1987, and Loma Prieta, 1989 earthquakes). In the meantime, the 1985 Mexico City earthquake unveiled relevant deficiencies in steel structures; however, it is beyond the scope of this paper. The second one is the major incident of the Northridge earthquake. The third one is the post-Northridge era, where the treatment actions were performed.

2.1 Pre-Northridge era

An influential point in starting to observe that the steel structures performed very well was the San Francisco earthquake in 1906. At this time, no welded moment-resisting frames (WMRF) existed. The proper behavior of the steel structures, as compared with the other construction materials, was also observed at the following major seismic events: 1925 (Sana Barbara), 1933 (Long Beach), and 1940 (Imperial Valley). However, all the previous events strengthened the view that steel is the best material to conform structures to earthquakes. It should be noted that the steel structures at this time were constructed by rivets or bolts with gusset plates conforming to and connecting build-up sections for beams and columns. This type of structure has the advantage of enhanced damping capacity and the disadvantage of pinched behavior.

With the advent of welding in the 1960s, as well as with the buildings becoming taller or having greater spans, hence requiring greater capacities, it was the moment that was introduced the welded beam-column connection Fig. 1 (the flanges of the beam are site welded with complete penetration welds, with the aid of backing bars, and a bolted shear tab with or without welds at the corners at the beam's web). From that timeframe, the great majority, of the moment-resisting frames in the US, was designed and constructed using the connection details provided in Figure 1.

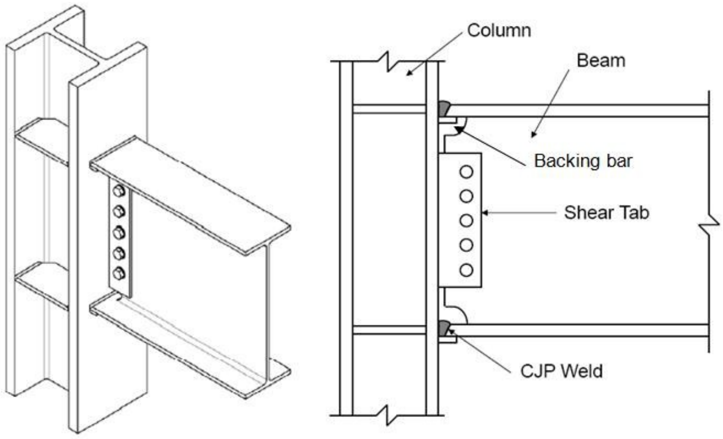

Fig. 1 Typical WMRF connection in the early 60s until 1994 Northridge earthquake.

The use of moment-resisting frames along with the specific construction detail was also validated by the fact that in the later earthquakes (e.g., Alaska, 1964; San Fernando, 1971; Whittier Narrow, 1987; and Loma Prieta, 1989), the behavior of the steel MRF constructions was excellent [3]. The 1990s were the culmination of credibility and technical reliability regarding the inelastic behavior of beam-to-column welded moment connections. Then, it was the time when, in the professional periodical of the American Institute of Steel Construction, AISC, appeared a competitive advertisement promoting the application of steel against concrete structures subjected to seismic actions Fig. 2 [4]. It was considered that structural steel, due to its inherent ductility, is safer than the other materials. This excellent behavior was also evidenced by the in-situ performance of steel WMRF.

2.2 The Northridge benchmark earthquake event

The Northridge earthquake (January 17, 1994), a seismic action with a relatively moderate magnitude of Mw = 6.7, unveiled the vulnerabilities of that type of structural system with this specific joint conformation (see figure 1). This happens, as with all things in real life, at the height of the trust that involved the moment-resisting frames with welded beam-to-column connections. The steel moment-resisting frames, generally welded with the E70ET04 electrode type, behaved in a brittle manner and not in a ductile manner, as was the main expectation. More specifically, the beam-to-column joint behavior was brittle and damaged in different ways, as presented in Figure 3 [5].

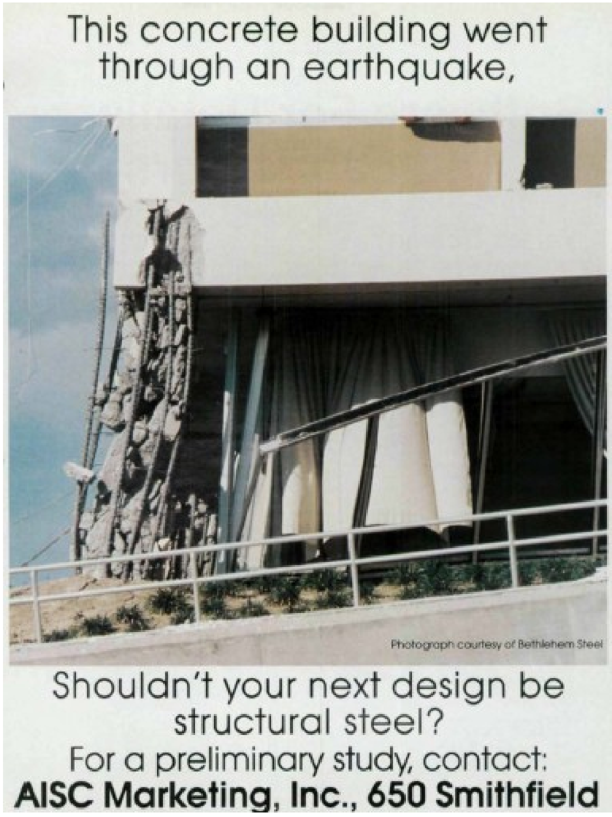

Fig. 2 An advertisement in Modern Steel Construction periodical which is encouraging the use of structural steel against concrete [4].

Initially, more than 150 buildings with no more than 10 stories high and welded moment-resisting frames were found to be damaged. Due to the difficulty of detecting the damage (because of fire protection or other types of architectural coverings), the City of Los Angeles issued a special ordinance for the inspection of steel frames within its jurisdiction; by the end of August, the damaged buildings were found to be 247 [6]. This type of brittle fracture was local, and the global safety of the buildings was assured. The limit state of life safety was integrally proven, with no collapse.

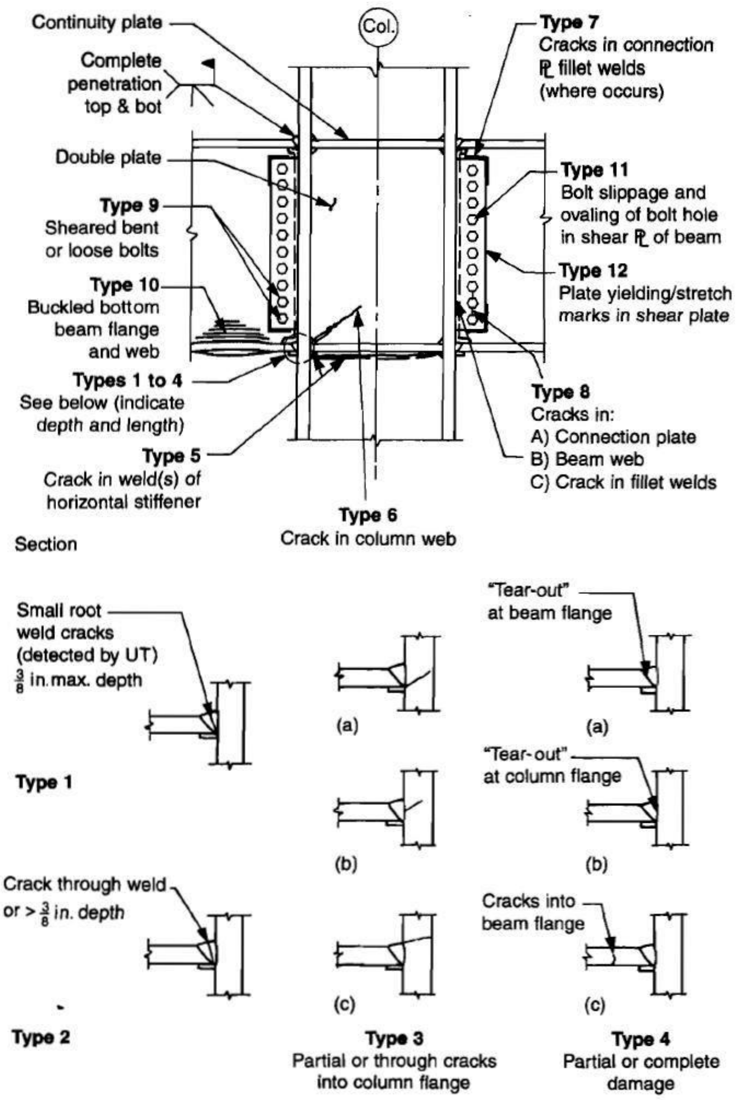

Fig. 3 Unexpected brittle damage at the welded beam-to-column joint [5].

It is important to mention that the San Fernando earthquake of 1971 pointed out mainly the inadequacies of the design practices related to the reinforced concrete structures and further on the near-field action (the pulse action, the effect of the vertical component, etc.).

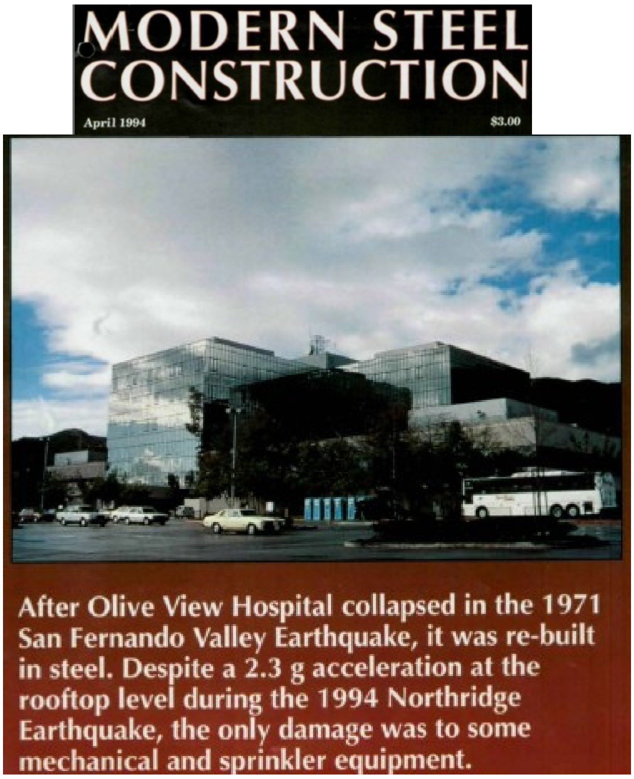

Fig. 4 The front cover of the Modern Steel Construction periodical promoting a strong message to recover the image of the steel structures [7].

2.3 Post-Northridge era

The US steel engineering community and building officials as well were surprised by this unexpected behavior. Moreover, exactly one year later, the Great Hanshin (Kobe) earthquake occurred on January 17, 1995, unveiling the same brittle fractures for moment-resisting frames, even in the case of a different type of beam-to-column connection. In mid-1994, US officials formed the SAC Joint Venture (SEAOC, ATC, and CUREE). The main goal was to investigate the damage to the welded steel moment-resisting frames, provide repair and strengthening techniques, and further improves the existing design and construction practices with a new framework in order to minimize such damage in future seismic events [8].

A radical change was performed after the Northridge earthquake; a new era for the design of steel structures and especially for the connections of moment-resisting frames was conceived. Before the Northridge earthquake, one can find one page related to the moment-resisting frames in the 1985 Uniform Building Code and four pages in the 1992 AISC edition for the seismic design. After the completion of the SAC research program in 2000, a series of guidelines (FEMA 351, 352, 353, 354, 355) and research reports were published for the aid of practitioners and the steel industry [9]. Based on this work, the American Institute of Steel Construction, AISC, developed the following standards: (i) the ANSI/AISC 341 (Seismic Provisions of Structural Steel Buildings), editions 2002, 2005, 2010, 2016, and 2022, and (ii) the ANSI/AISC 358 (Prequalified Connections for Special and Intermediate Moment Resisting Frames for Seismic Applications), editions 2005, 2010, 2016, and 2022 [10]. Moreover, the American Welding Society published the AWS D 1.8 (Seismic Supplement), which focused on materials, connection details, workmanship, and inspection issues. Nowadays, prequalified moment connections are completely codified, including non-proprietary and proprietary connections Fig. 5.

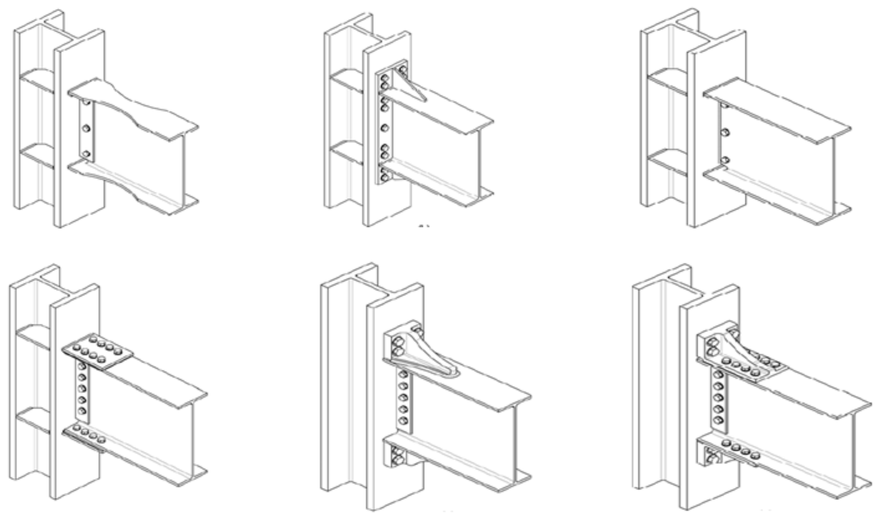

Fig. 5 A sample of prequalified connections according to ANSI/AISC 358 [10].

3 Qualitative analysis through the use of the SCAM

According to J. Reasons Swiss Cheese Model [1], in a complex system against an accident, we distinguish (i) the defenses, barriers, and safeguards, where they present the layers of protection; (ii) each layer of protection has holes, some where they can be the active faults (generally attributed to human fallibility); and others are characterized as the latent conditions (latent or dormant), that are attributed to the system's inefficiencies, let’s say organizational, design-related, or ineffective communication, training, inadequate inspection, and supervision. At the moment when all the holes in the protective layers are aligned, an accident is going to occur (Fig. 6). Two aspects are important to mention: Firstly, according to Turner's theory [11], before an accident, there is an incubation period where all the facts are developed and accumulated unnoticed. Secondly, an accident is a combination of factors that breach the protective layers.

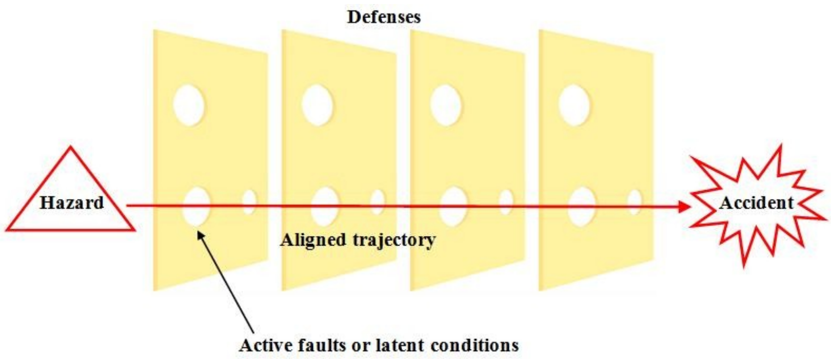

Fig. 6 A schematic representation of the J. Reasons' Swiss Cheese Accident Model

The aforementioned model is based on real facts, attitudes, procedures, and human fallibility in any kind of action; in our case, it takes into account the historic background, the past experimental evidence, the evolution of design and construction practices in the USA, and the evolution of codes as well. For instance, the strong believe and faith of suitable performance of MRF until the Northridge earthquake, the relaxed, of the poor connection detail and generally the constructional conformation, leading to extreme reduction of redundancy and extreme ductility reduction, the relaxed welding inspection, the incompatibility between the experimental research and real application practices, the not well accounted-understood effects of near field action (e.g. vertical component, strain rate, forward directivity, pulse characteristic action), against the barrier which is the evolution of the Uniform Building Code, UBC, from '60s until to '80s, (and further on the AISC 1992 edition) depicts such a path until the famous brittle fractures observed in the Northridge earthquake. In addition, two unnoticed facts should be remarked: the first one is the two brittle failures of WSMRF observed at the San Fernando earthquake in 1971 on buildings under construction, and secondly, the undetected brittle failures of at least five buildings that were caused at the Loma Prieta earthquake in 1989, which were detected after the Northridge earthquake [6]. The first one was alarming evidence that was not taken into account, while the second one illustrated the unsatisfactory policies of post-earthquake inspection related to the steel moment-resisting frames. Both of them are a combination of active and latent conditions that occurred during the incubation period.

Focused on the causes that provoked the brittle fractures on WSMRF, the US engineering community did not unanimously accept a certain couple of factors that are attributed to the unexpected failures. Therefore, it is important to briefly describe the main factors observed for the in situ and post-Northridge experimental performance of WSMRF connections: (i) welding-related issues, concerning undetected welding factors such as low weld metal toughness, poor quality welding, inadequate workmanship, inadequate inspection, (ii) poor connection detail with very high localized stress and strain at the column-connection face, due to improper detailing, (iii) steel material issues, where a higher actual yield stress than considered in design altered the strong column-weak beam in a weak column-strong beam; (iv) seismic load-related issues through the near field effect; (v) structural configuration-related issues through the reduction of structural redundancy by using also perimeter WSMRF while the other connections are pinned; the coupling of steel beam-slab effect, which also altered the capacity design conditions for a strong column and weak beam. It really was the combination of all the technical factors, or more than three of them, that led to the unforeseen damage.

In each case, without the cause, there is no effect; without a hazard, there is no accident. Focusing specifically on the hazard, such as the near-field action with its distinct characteristics, it is evident that the reinforced concrete moment-resisting frames were significantly compromised during the 1971 San Fernando earthquake due to unexpected near-fault effects. Despite this early warning, captured by researchers like Prof. V. Bertero [12], regulatory actions were not implemented. In the 1970s, particularly after the San Fernando earthquake, welded steel moment-resisting frames (WSMRF) gained popularity over reinforced concrete frames. However, these steel frames also exhibited vulnerabilities linked to the unique aspects of seismic activity, including strain rate, strong velocity pulses, and the impact of the vertical earthquake component. The prevailing assumption was that steel's ductility alone ensured safety against all types of seismic actions. The research and engineering communities continued to use for the experiments loading protocols that were based on cyclic repetitive actions (e.g., a far-field earthquake), to perform inelastic analysis without considering the severity of seismic action within approximately 10 km from the epicenter, and to design without considering column axial and beam flexural demands, especially in interior structural elements and upper stories, as well as the increased action of the vertical component. It was a reluctant consideration. All the aforementioned are the holes in the protective layers at different defense levels. The near-field action was the trigger to expose the vulnerability of WSMRF. A year later, the Kobe earthquake in 1995 also unveiled the same unsatisfactory behavior, even in the case that the Japanese connection details and practices are different from those in the US. The hazard will exist, and the only action is to fill the holes by taking into account any technical information coming from each new seismic ground motion and, further on, to implement building regulations, which represent one of the main safeguards against any collapse.

Starting to follow the trajectory from the pre-Northridge era to the Northridge earthquake incident, one can distinguish the next plane of action according to the Swiss Cheese Accident Model (Fig. 7). (i) Mainly, the first hole is related to experimental testing and, in a lesser way, to analytical studies. In the great majority of the experiments that had been carried out, in one way or another, the trend of the welds toward brittle fracture had become apparent [6]. Already in the first experimental program that implemented cycle loading, Popov and Pinkey (1969) pointed out the importance of welding quality and the corresponding inspection. Moreover, they also reported the importance of the size effect [6]. The experimental work of Engelhardt and Husain (1993), published a month before the Northridge earthquake, also warned about weld quality and control [6]. Early warning signals are ignored completely and translated wrongly. It is noteworthy to note the following issue: the experiments conducted in the laboratory regarding welding conditions, beam-column section sizes, and slab effect have nothing to do with real construction conditions. This was a technical, communicative, and administrative failure. It was the beginning of the end. The second hole is related to the regulations (codes, standards, and specifications). The building officials and the professional bodies who are responsible for the drafting of regulations did not capture and embody the results provided by academic and industry research. At some point, they are constrained by other than technical reasons (e.g., political, financial, or industry interests) [6]. The 1959 Blue Book (SEAOC, 1959) considered as a dogma the ductility of steel structures. The 1968 Blue Book commentary (SEAOC, 1968), fortunately, recognized and imposed material conditions ensuring the ductility of steel frames. Moreover, this edition of Blue Book provides the first attempt at codifying WSMRF (introducing requirements for ductility and quality control in the welded joints). The 1976 UBC (Uniform Building Code, 1976) imposed stringent measures for welds' inspection [6]. The 1988 UBC defined the capacity design (strong column-weak beam). Finally, one of the major professional bodies representing the steel industry, the American Institute of Steel Construction (AISC), published the 1992 AISC, the first standard related to the seismic design of the welded special moment resisting frames. There was technical information, but it was not fully appreciated and was sometimes ignored. It is clear that the reporting and monitoring system for the evolution of the building regulations did not work very well; there was a lack of systematic technical overview. It is well known that in the USA there was not a harmonized framework regarding the structural design regulations; only in the 1990s and early 2000s did radical changes occur [13]. The third hole is related to the design and construction practices. The fundamental rule of thumb is to weld in the shop and bolt at the site. It violated a rule that generally can avoid or mitigate poor welding workmanship. Accordingly, every inspection process is undermined due to the fact that the welding is influenced by the environmental conditions, welder's capacity, position, and attitude; *´´the weld quality is into the product ´´* and not at the inspection procedure. Moreover, during this time, designers reduced the redundancy by using SWMRF only at the periphery and/or at selected points of the plan configuration and pin connections for the interior frames. Overall, the final safety net to ensure safety is the administrative controls and the applied policies of building jurisdictions. Until the Northridge earthquake, it was not so active; after the Northridge earthquake, they emitted guidelines for the design, construction, and certification of welded connections in new steel buildings.

In the post-Northridge period, experimental testing, theoretical analysis, and numerical analysis provided by the SAC Joint Venture Research Program scrutinized the performance of the WSMRF, leading to a new era for the design and construction of steel. The ill-defined issue of welded moment connections was resolved with the prequalification of different types of connections (bolted, welded, reduced beam moment connections, proprietary joints) (ANSI/AISC 358). Further on, it was also regulated in the design of steel structures subjected to seismic loading (ANSI/AISC 341). Currently, the holes are filled (Fig. 8).

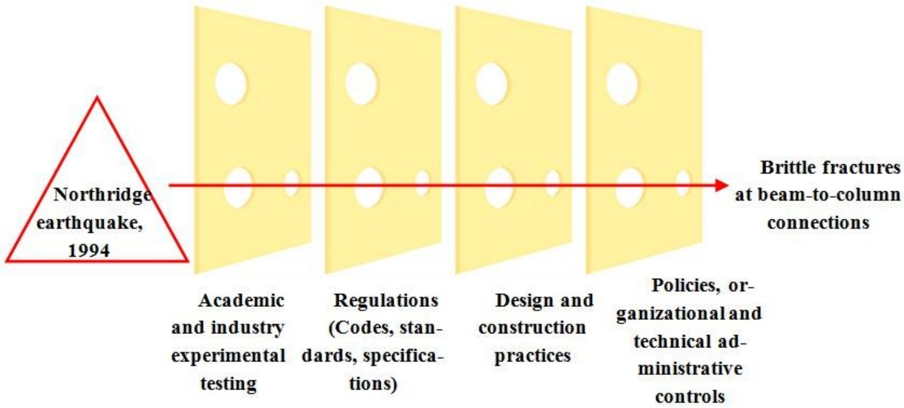

Fig. 7 Swiss Cheese Model application for the damage in welded connections after the Northridge earthquake, 1994.

The California Assembly Bill AB 2681, which was vetoed in September 2018, declared that the design and construction of the buildings that were approved by the city or county under the 1995 or earlier edition of the California Building Code are potentially vulnerable. Recently, the AISC published ANSI/AISC 342-22 (Seismic Provisions for Evaluation and Retrofit of Existing Steel Buildings), which fills the gap on how to rehabilitate the existing stock of steel buildings that suffered potential damage from past earthquakes.

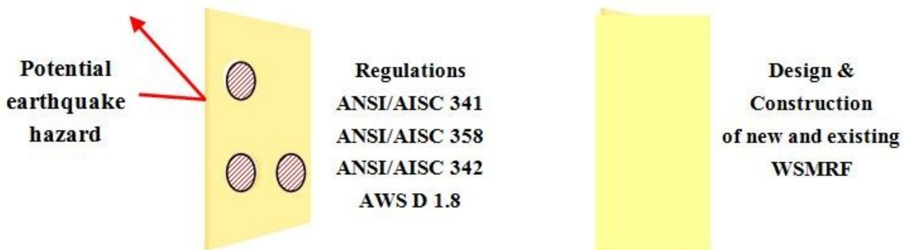

Fig. 8 Swiss Cheese Model application for the brittle post Northridge earthquake period.

4 Conclusions

The unexpected brittle fracture in the WSMRFs was a collective engineering failure and had a great impact on society. The damage to the WSMRF was not only a technical failure but also, in equal measure, an institutional and administrative failure. It was revealed that the results of the academic research were not taken into consideration by the industry, California State, or even the government. The alarming signs for the quality of welding and member extrapolation with caution were never taken into consideration by the professional and regulatory bodies or by designers. On the other hand, neither academic research nor construction efforts took into account real-world conditions. Really, it is a permanent hole in the construction system. In many cases, academia and industry follow parallel paths that do not intersect. However, an exemplary practice was the SAC Joint Venture Research Program, where all the stakeholders (academy, industry, and government) put in their best efforts, and the results were an evolutionary change in the US steel construction industry. It is imperative for the safety and evolution of the regulations (codes, standards, specifications, and guidelines) to implement a rigorous system that monitors, collects, reports, and publishes all the developments and advancements in the corresponding field. An active and permanent review panel, not a bureaucratic one, must be developed in regulatory bodies; the target is to capture all the new research, collect information from practitioners, learn from failures, and follow the application of regulations. Therefore, barriers to the failure of the construction system present the need for the development of proper policies that connect academic and industry research with professional bodies and public institutions.

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