# 2010 MAULE CHILE EARTHQUAKE WALL PERFORMANCE AND ITS APPLICATION TO IMPROVEMENT OF THE AASHTO LRFD SEISMIC WALL DESIGN SPECIFICATIONS

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## Abstract

The 2010 Maule Chile earthquake provided a natural full scale laboratory to investigate the seismic performance of walls designed using the AASHTO design specifications (AASHTO 2002). Examples of wall performance in Chile during that earthquake and lessons learned, as well as wall performance observations from other earthquakes and seismic research, were used to develop the first major update of the AASHTO specification seismic wall design sections in almost 20 years. Specification improvements described include a no seismic design option, recommendations for wall design and construction details to improve seismic performance, and a revised seismic earth pressure design approach, resulting in more cost effective wall designs.

## Introduction

On February 27, 2010, a magnitude 8.8 earthquake occurred just off the coast of the Maule region in central Chile, affecting the central valley and coastal areas, and strongly affecting Chile's two largest cities, Santiago and Concepción. This was a major subduction zone earthquake, a type of earthquake that is not uncommon in Chile. A reconnaissance team (including the author) organized by the US Federal Highway Administration (FHWA) investigated the performance of Chile's transportation infrastructure shortly after the earthquake (Yen, et al., 2011).

This paper summarizes the performance of the retaining walls in that earthquake and identifies lessons learned. Considering the magnitude of this earthquake and the observed good performance of walls, even those close to the epicenter, it was decided to use the information gained to help evaluate the potential for developing improvements to the AASHTO LRFD Bridge Design Specifications regarding seismic wall design. This paper also describes how this information, seismic wall performance information from other earthquakes, and seismic research was used to develop the first major revision to the AASHTO LRFD seismic wall design specifications (AASHTO 2012) in almost 20 years.

## 2010 Maule, Chile Ground Motions

Ground motions from this earthquake were felt strongly from Santiago (335 km NE of the epicenter) to the Arauco Peninsula over 100 to 150 km south of the epicenter. Peak ground accelerations ranged from 0.17g to 0.3g at good soil sites, but to as high as 0.56g in poor soil sites, in the Santiago vicinity and in the central valley. In Concepción on the coast, peak horizontal and vertical ground accelerations of 0.6g or

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more were observed.  One of the most notable features of this earthquake was its duration.  Figure 1 provides an example ground motion in the Concepción area and illustrates its duration.  The ground motions experienced in Chile in this earthquake are likely to be similar to what the coastal areas of Washington and Oregon in the USA could experience in the future and represent the design level event for that area.

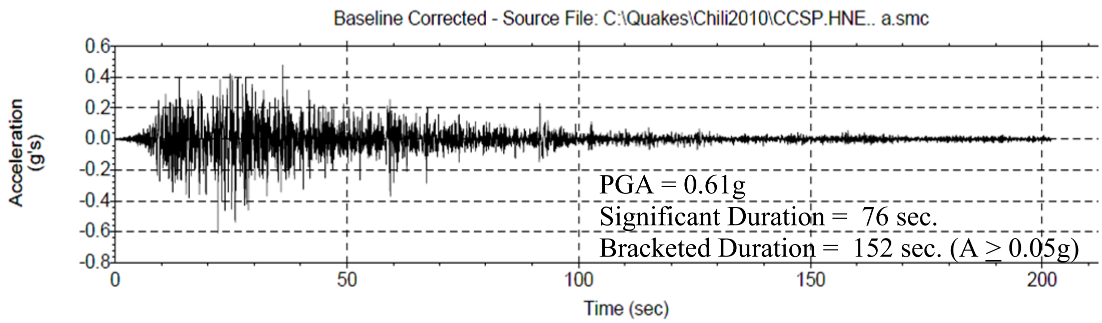


**Figure 1.  Example Ground Motion from San Pedro De La Paz, Concepción (after Yen, et al. 2011).**

## Wall Design in Chile

Transportation infrastructure walls in Chile have been designed using the AASHTO Standard Specifications for Highway Bridges (AASHTO 2002) since the mid-1950's.  Therefore, this earthquake provided an excellent opportunity to evaluate how walls designed using the AASHTO specifications could perform in a design level earthquake in the Pacific Northwest of the United States (US).

As is true for bridges, design peak ground accelerations (PGA's) for walls in Chile were only 0.12g before 1985, increasing to 0.15g after 1985, and increasing again in 2001 to 0.4g on the coast and 0.3g in the central valley (Yen, et al. 2011).  The design acceleration is reduced to 50% of the PGA for gravity walls, allowing some movement of the wall to occur, though for reinforced soil walls, this reduction in the acceleration was not allowed.

## Wall Performance in Chile

Wall performance in the 2010 Chile earthquake was observed at 14 sites, and each site typically had multiple walls.  Wall types evaluated included panel and concrete block faced reinforced soil walls, concrete gravity walls, and anchored walls. Wall heights ranged from a few meters to over 12 m.  Backfill soil for most walls was granular, ranging from fine uniform sands to well graded gravels.  Overall, wall performance was very good, with only limited damage, and no walls collapsed.  The first three figures (figures 2 through 4) show examples of walls that performed well. The next four figures (figures 5 through 8) illustrate walls that suffered damage but that did not collapse.

Figure 2(a) shows two walls retaining the approaches to the Americo

Vespucio/Independencia Bridges in Santiago, built in 2004. The wall to the left in the figure is a high density polyethylene (HDPE) geogrid reinforced wall with a dry cast concrete block facing, and the one on the right is a steel reinforced soil wall with concrete precast panels. Both walls are approximately 6 to 7 m in height. Figure 2(b) shows the bridges at the same interchange, which did sustain some damage that required the bridge to be shored until repairs could be made. Other than some toppling of facing blocks at the top of the block faced wall due to poor connection and coping details, the walls showed no signs of damage due to the earthquake.


**Figure 2. The Americo Vespucio/Independencia Interchange Showing (a) Bridge Approach Walls, and (b) the Bridges and Abutments.**

Figure 3 shows an exceptionally large (i.e., approximately 12 m high) concrete faced gravity wall supporting the Maipu River Railroad Bridge just south of Santiago. Both the wall and bridge were not damaged and were fully operational after the earthquake, yet parallel bridges at this site did suffer significant damage, but without collapse.

Figure 4(a) shows one of the dry cast concrete block faced HDPE geogrid reinforced soil walls that form the bridge abutments for the Avenida Independencia Bridge at Estribo Francisco Mostrazal between Santiago and Rancagua in the central valley. These walls, 7.4 m in exposed height, directly support the bridge abutment footing load of approximately 210 kPa at each abutment, the typical maximum allowed footing loading in the AASHTO design specifications. The walls were designed using the 2002 AASHTO Standard Specifications (Tensar Earth Technologies 2003). For seismic design, the acceleration coefficient used was 0.3g. Though ground motion data was not available at this particular site, based on ground motion data in the general vicinity, actual PGA's were likely in the range of 0.3g to 0.5g. Reinforcement spacing varied from 0.2 to 0.4 m, and reinforcement length varied from 80 to 100% of the wall height. Most of the reinforcement layers had an ultimate tensile strength of approximately 130 kN/m. Good quality crushed stone backfill ($42^{o}$ design friction angle) was used for the walls.

As can be seen in the figure, the abutment walls showed no sign of damage or permanent deformation. However, the bridge superstructure was damaged, but still functional, as shown in Figure 4(b). The bridge superstructure has both significant skew and a significant longitudinal downward slope. The combination of slope and skew caused the bridge superstructure to move downslope and toward the acute angle in the bridge skew, resulting in the damage, yet the walls were unaffected. This case history is a testament to the ability of reinforced soil wall structures to resist large earthquake loading.

**Figure 3. Abutment Wall Supporting the Maipu River Railroad Bridge South of Santiago.**

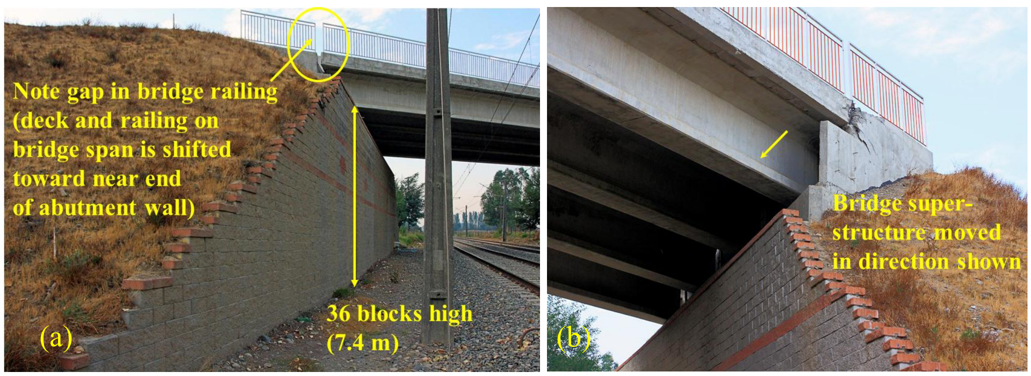


**Figure 4. Avenida Independencia Bridge Geogrid Reinforced Soil Walls at Estribo Francisco Mostrazal (a) West abutment, and (b) East Abutment.**

Figure 5 shows one of the first reinforced soil walls built in Chile, in 1995. This wall is located in Concepción, near one of the hardest hit areas in that location. The site is underlain by 6 to 7 m of loose sand and silt with a high water table, and some sand boils (evidence of liquefaction) were observed. One of the older bridges at the site collapsed, and the other bridges were significantly damaged (Yen, et al. 2011). The wall shown in the figure is approximately 10 m high where it makes contact with the severely skewed Via Elevada railroad undercrossing bridge abutment wall. The severe skew restricted the length of the steel soil reinforcement (bar mats) attached to the wall facing panels closest to the bridge abutment wall. Furthermore, the vertical joint between the reinforced soil wall and the bridge abutment wall was not tied together to prevent movement and separation at the vertical joint, though in all other respects, the wall was designed in accordance with the AASHTO bridge design specifications available at that time, but using a higher seismic design acceleration (0.4g) than required by Chilean design standards at that time (W. Neely, Aug. 30, 2010 personal communication). The backfill used for the wall was a uniform medium river sand. Based on testimony from the wall supplier (W. Neely, personal communication), the backfill soil used was very difficult to maintain compaction, causing difficulties in keeping the facing panels aligned during wall construction. As can be seen in Figure 5, the wall top moved outward about 0.3 m, likely the result of the lack of connection at the vertical joint combined with the medium uniform sand backfill.


**Figure 5. Abutment and Reinforced Soil Wall at the Via Elevada Railroad Undercrossing.**

The other walls at the bridge shown in Figure 5 and other bridges at this interchange exhibited similar performance problems where the reinforced soil walls joined with the bridge abutment walls. An example of the walls at these other bridges is shown in Figures 6 and 7. In this case, the lower wall moved outward over 0.3 m, due to a combination of lateral sliding and outward rotation. It is possible that liquefaction induced weakening of the soil below the wall contributed to this movement. However, of all the walls observed in Chile, this is the only wall that exhibited what appeared to be a sliding failure. In all other cases, when wall movement occurred, it was due to rotation of the wall, or liquefaction induced displacement. As the wall moved outward, a large gap formed due to the lack of connection across the vertical joint, allowing much of the sand backfill to flow through the opening.


**Figure 6. Tiered Walls at the Via Elevada Railroad Crossing (a) Overall View of Walls, and (b) Close-up Showing Movement of Lower Tier.**

Figure 8 shows one of the four dry cast concrete block faced HDPE geogrid walls built in 2009 that retain the approach fills for this railroad undercrossing near Talca in the central valley, almost directly east of the earthquake epicenter. PGA's in this area were likely in the range of 0.5g, based on ground motion records in the general area (e.g., Curico) and its relatively close proximity to the epicenter. The walls have a maximum height of over 9 m plus a 2 m high surcharge. Another important feature of these walls is the very tight curve in the wall alignment. Based on construction photos, geogrid strips were placed throughout the curve, requiring the geogrid strips to overlap one another through the curve. It was not clear if some soil was placed between the overlapping geogrid strips to ensure good pullout resistance, and how well the geogrid connected with the facing blocks within the curve in the wall alignment.


**Figure 7. Wall Shown in Figure 6, but from Back Side Showing Exposed Junction between Reinforced Soil Wall and Bridge Curtain Wall.**

As can be seen in this figure, the wall shown did exhibit significant deformation, primarily through tilting, as no sliding of the wall at its base was observed. The tilting of the wall resulted in the formation of 45$^{o}$ shear bands (see especially Figure 8c) in the facing blocks, especially transverse to the bridge centerline and in the vicinity of the tight radius curve in the wall alignment. Some deformation and shearing also was present in the other four walls at this site, but not as severe as the wall shown in the figure.

## Summary of Wall Performance in Chile and Lessons Learned

Overall, very few walls within the region affected by the 2010 Maule earthquake exhibited even minor damage, and no wall actually collapsed (Yen, et al. 2011). This observation applied to reinforced soil walls, concrete gravity walls, anchored walls, and soil nail walls (Yen, et al. 2011). Of the walls that did exhibit some damage, as illustrated in the previous figures, the damage typically consisted of a few toppled facing blocks, separation of vertical joints between the wall and adjacent structure, tilting of the facing, though generally less than 0.2 to 0.3 m, and in one case, shearing of the facing blocks. With the exception of the walls that were subjected to liquefaction effects, most of the wall performance problems observed were due to marginal or inadequate design details that could be improved. Walls generally did not exhibit basal sliding or settlement unless the soil below the wall exhibited liquefaction, and even in that case, no wall collapse was observed. This may indicate that sliding resistance is much greater than assumed in design. This finding is similar to that described by Koseki, et al. (2006) with regard to observations from model walls in shake table studies.


**Figure 8. Muros Talca Railroad Crossing with Dry Cast Concrete Block Faced HDPE Geogrid Walls (a) Ground View, (b) View of Wall from Bridge, and (c) Close-up of Wall Face Showing 45º Shear Bands.**

A lesson learned from the performance of walls in Chile is the importance of using good details for wall design and construction. Specifically, the following should be considered for future design of walls in seismically active areas:

- Avoid uniform sand backfill, especially if it lacks angularity. While uniform sand is well drained, it can be unstable with regard to reinforced soil wall backfill, allowing soil reinforcement to slip and facing panels to separate. A more well-graded mix of soil, but with low silt/clay content, is preferred.

- The top facing blocks or panels should be tied together well to prevent toppling. This can be accomplished through the use of good coping details, and possibly connecting and grouting the top few blocks or panels together.

- Wall corners and tight radius curves in the wall alignment tend to exhibit damage more often than relatively straight sections of wall. If wall corners are not properly joined together, they can open up and allow backfill to spill through during shaking. In severe cases, shearing of panels or facing blocks can occur. Special details may be necessary at corners or sharp turns in the wall alignment to prevent separation of panels and to address the three dimensional aspects of the corner or tight radius curve.

- Full height joints in walls can come apart during shaking. These joints should be structurally tied together adequately to prevent separation during shaking, but not so tight that the joints attract load and differential settlement issues cannot be addressed.

- For reinforced soil walls, a minimum reinforcement length of 70% of the height or more, especially in upper part of wall, appears to prevent excessive lateral deformation of wall face. This is especially important for heavily skewed bridges where the reinforced soil wall joins the curtain wall or abutment wall.

## Application of Lessons Learned to Improve the Seismic Wall Design Specifications

Recognizing the very good performance of walls in the 2010 Maule earthquake, considering the severity of the ground motions, a reasonable next step is to re-evaluate the seismic wall design provisions of the AASHTO design specifications. To make sure such specification changes are broadly applicable, wall performance in other earthquakes and seismic wall research were considered. Furthermore, research conducted at the request of the AASHTO Bridge Subcommittee (Anderson, et al. 2008) was considered, as it was the Subcommittee's intention to use the research results provided in that report as the basis for updating the seismic wall design portions of the AASHTO LRFD Bridge Design Specifications. The AASHTO T-15 Technical Committee took the lead in implementing this research plus the lessons learned from Chile to complete the revisions needed. Since the AASHTO specifications are national in nature, national (i.e., US) input from the voting members of the Bridge Subcommittee (i.e., all state transportation departments), academia, and the wall industry was obtained, including review of draft design specification changes. Only the development background for some of the key changes made to the seismic wall design articles in the AASHTO Specifications are described herein due to paper length restrictions. Specifically, the following are addressed: (1) the development of no seismic analysis provisions for walls, (2) improvements to the approach used to estimate seismic earth pressure, and (3) the development of recommended wall details for improved seismic performance.

The concept of a no seismic analysis option was initially developed by Anderson, et al. (2008). The criteria they recommended were very simple, considering the peak ground acceleration and slope above the wall. This was used as a starting point. To establish a no seismic analysis provision for walls, key criteria to determine whether or not a no seismic analysis is allowed for a given wall were needed. Criteria selected to be developed for this purpose are as follows: (1) the maximum acceptable PGA, (2) potential for liquefaction or presence of sensitive clays, (3) the wall application, and (4) total wall height.

Regarding the first criterion, maximum PGA, the concept is to require wall seismic design only for cases in which there is a risk of poor wall performance. Opinions varied widely among the US states regarding the maximum allowed PGA, as the value selected defines which US states must do seismic design of walls. Recommendations from various researchers regarding the value selected also varied. Even as early as 1970, Seed and Whitman (1970) concluded that "many walls adequately designed for static earth pressures will automatically have the capacity to withstand earthquake ground motions of substantial magnitudes and in many cases, special seismic provisions may not be needed," and further indicated that this applies to gravity walls with PGA's up to 0.25g. Anderson, et al. (2008) recommended using a PGA of 0.2 to 0.3g, depending on the soil slope above the wall. More recently, Bray, et al. (2010) and Lew, et al. (2010a, 2010b) indicate that lateral earth pressure increases due to seismic ground motion are likely insignificant for PGA's of 0.3g to 0.4g or less, indicating that walls designed to resist static loads (i.e., the strength and service limit states) will likely have adequate stability for the seismic loading case. Figure 9 shows the results of centrifuge modeling of walls under seismic loading conducted by Al Atik and Sitar (2010) that illustrates seismically induced lateral earth pressure does not become significant until the PGA exceeds 0.4g.

A PGA criterion can also be developed empirically from wall performance in past earthquakes, such as the 2010 Maule Chile earthquake. Clough and Fragaszy (1977) assessed damage to floodway structures, consisting of reinforced concrete cantilever (vertical) walls structurally tied to a floor slab forming a continuous U-shaped structure, due to the 1971 San Fernando earthquake. They found that no damage was observed where PGA's along the structures were less than 0.5g (see Figure 10). However, damage and wall collapse was observed where accelerations were higher than 0.5g or where the structures crossed the earthquake fault, though in the latter case damage was quite localized. They noted that while higher strength steel rebar was used in the actual structure than required by the static design, the structure was not explicitly designed to resist seismic loads. Gazetas, et al. (2004) for concrete gravity walls in the 1999 Athens earthquake and Lew et al. (1995) for tieback shoring walls in the 1994 Northridge earthquake observed that wall performance was good for peak ground accelerations up to just under 0.5g even though the walls were not specifically designed to handle seismic loads. AASHTO (2012) provides a more complete summary of wall seismic performance in Appendix A11.

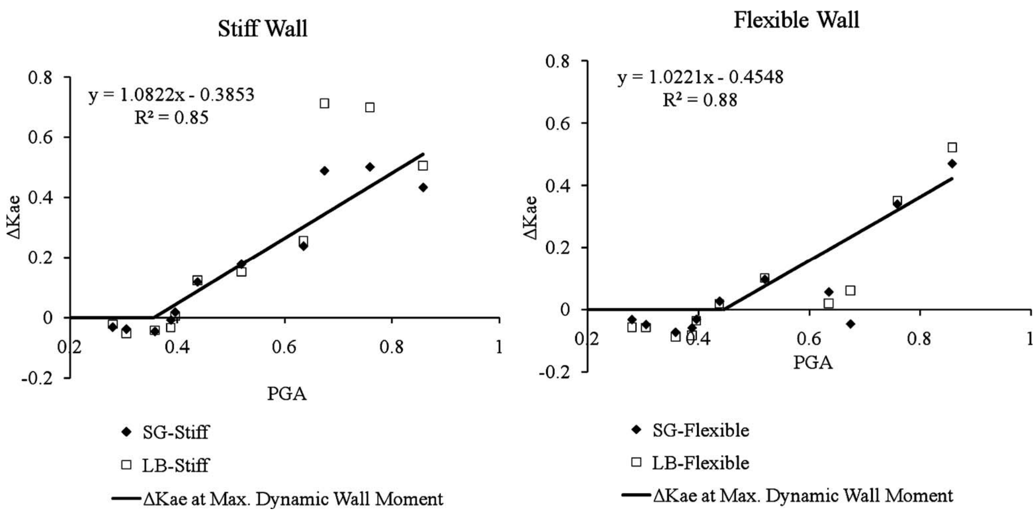


**Figure 9. Centrifuge Wall Modeling Results Illustrating when Dynamic Earth Pressure ($\Delta K_{ae}$) Becomes Measurable (after Al Atik and Sitar, 2010).**

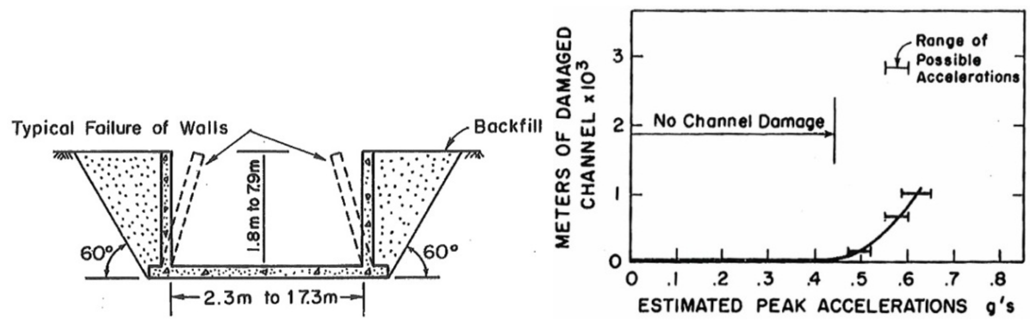


**Figure 10. Floodway Walls in the 1971 San Fernando Earthquake, (a) Floodway Wall Configuration and Failure Mechanism, and (b) PGA versus Length of Walls Damaged (after Clough and Fragaszy 1977).**

While there have been some notable wall failures in past earthquakes, those failures were for situations where more severe ground motions that noted above occurred, severe liquefaction occurred, or for much older walls that did not come close to even meeting current design standards for static loading. For example, Seed and Whitman (1970) indicated that severe displacement or collapse occurred for some concrete gravity walls and quay walls in the great Chilean Earthquake of 1960 and in the Niigata, Japan Earthquake of 1964 due to severe liquefaction. But at sites where the water table was deeper, severe wall damage or collapse was an infrequent occurrence. Tatsuoka, et al. (1996) indicated that collapse occurred for several of the very old (1920's to 1960's) unreinforced masonry gravity walls and concrete gravity structures exposed to strong shaking (e.g., as high as 0.6g to 0.8g) in the 1995 Kobe Japan earthquake, likely due to the presence of weak foundation soils or the presence of a very steep sloping surcharge (e.g., 1.5H:1V) combined with poor soil conditions.

As summarized previously, wall performance in the 2010 Maule earthquake was very good, even for relatively high long duration earthquake ground motions, though in most cases those walls were designed for seismic loading. If problems did occur, it was typically due to wall details that could be improved to ensure good seismic performance. Furthermore, some of the tallest walls did suffer more significant damage, but without collapse.

Therefore, from these observations combined with available research, the following can be concluded with regard to a wall no seismic analysis provision:

- Setting the limit at 0.4 g for a no seismic analysis provision represents a reasonable compromise between observations from laboratory modeling and full scale wall situations (i.e., lab modeling indicates seismic earth pressures are very low below 0.4g, and walls in actual earthquakes start to have serious problems, including collapses even in relatively good soils, when the acceleration is greater than 0.5g and the wall has not been designed for the seismic loading).
- Liquefaction can contribute to severe wall damage, displacement, and possibly collapse. Therefore, if significant liquefaction can occur, a seismic analysis

should always be conducted. This also applies to walls supported on sensitive or otherwise already weak cohesive soils.

- If wall damage occurs, it appears more likely to occur with taller walls, say 10 m or more in height. Therefore, seismic analysis should normally be conducted if the wall height is approximately 10 m or more.
- Walls that are in critical applications, such as those that support other structures, should always be evaluated for seismic loading, considering both the stability and performance of the wall itself, and the effect the wall performance has on the structure it supports.
- The wall details are important for obtaining good seismic performance, especially how abrupt changes in wall geometry and vertical joints are tied together, and how the top facing elements are tied together (e.g., with the coping). In 2010, the AASHTO specifications did not have much information on what are considered good seismic details for walls. Therefore, good seismic details should be a condition of use of the no seismic analysis option for walls.

Considering the good performance of walls typically observed in even the largest and most damaging of earthquakes, it is likely that the methods currently in use to estimate the seismically induced lateral earth pressures on walls are conservative. Therefore, factors contributing to design conservatism in the seismic earth pressures used for design are investigated. The Mononobe-Okabe (MO) method to estimate seismic earth pressure has been in use for many years and is likely one source of this conservatism, especially in certain situations (Koseki, et al. 1998; Nakamura, 2006). The MO method, while simple to use, also has limitations in its applicability, and due to lack of alternative design tools, has been used in situations for which the method was not intended (AASHTO 2012). Anderson, et al. (2008) developed an alternative Generalized Limit Equilibrium (GLE) Method that could be applied, with greater accuracy, to a wider range of situations, such as layered soils, high ground water, and the presence of soil cohesion. This method will yield similar results to the MO Method for the simpler situations for which the MO Method was developed, as the theoretical basis for both methods still results in flattening of the failure surface and increase in failure mass due to seismic acceleration, contributing to conservative predictions from both methods (Koseki, et al. 1998). However, the GLE Method can be used to advantage to account for factors the MO Method is not capable of addressing, reducing the level of conservatism in those situations. Therefore, the Anderson, et al. (2008) alternative procedure is now included in the AASHTO design specifications and the limitations of the MO Method are described. While this is a step in the right direction, the MO analysis method improvements to seismic earth pressure prediction developed in Japan (Koseki, et al. 1998) should, however, be considered in the future for the AASHTO seismic wall design specifications.

Another source of conservatism in the prediction of seismic earth pressure is the treatment of the active soil mass behind the wall during shaking as a rigid body (Koseki, et al. 2006). This affects how wall deformation during shaking affects the seismically induced forces in the wall and the soil mass and the assumption that the seismic forces act simultaneously on the wall and the backfill. In the past, this issue has been crudely addressed through reduced acceleration theoretically calculated using

Newmark analysis, and in some cases with discounting a portion of the seismic active earth pressure when combined with the wall inertial force, at least for reinforced soil walls (AASHTO 2002). Anderson, et al. (2008) partially addressed this though a recommended wave scattering factor and improvements to prediction of seismic earth pressure reduction as a function of sliding deformation. While these improvements are still somewhat dependent on the rigid body assumption, they do reduce some of the conservatism in the prediction of seismic earth pressure. These recommended improvements are now included in the new (AASHTO 2012) design specifications. Refinements, based on Nakamura (2006), in how seismic earth pressure forces are combined with wall mass inertial forces to account for phase differences in these forces have also been developed and included in the AASHTO specifications.

The location of the seismic earth pressure resultant can be another source of conservatism in assessing the seismic stability of walls. Mononobe and Matsuo (1929) originally suggested that the resultant of the active earth pressure during seismic loading remain the same as for when only static forces are present (i.e., H/3, where H is the wall height). However, theoretical considerations by Wood (*1973*), who found that the resultant of the dynamic pressure acted approximately at mid-height, and empirical considerations from model studies summarized by Seed and Whitman (1970), resulted in increasing the height of the resultant location above the wall base to H/2, which has been used for many years in US design practice. Back analysis of full scale walls in past earthquakes, however, indicates earth pressure resultants located higher than H/3 will overestimate the force, resulting in a prediction of wall failure when in reality the wall performed well (Clough and Fragaszy, 1977). This appears to be consistent with observations of walls in Chile. Recent research also indicates the location of the resultant (static plus seismic) should be at H/3 based on centrifuge model tests on gravity walls (Al Atik and Sitar, 2010, Bray, et al., 2010, and Lew, et al. 2010). However, Nakamura (2006) also indicates that the resultant location could be slightly higher, depending on the specifics of the ground motion and the wall details. A reasonable approach is to assume that for routine walls, the combined static/seismic resultant should be located at the same location as static earth pressure resultant, but no less than H/3. However, a slightly higher resultant location (e.g., 0.4H to 0.5H) should be considered for walls in which the impact of failure is very high.

With regard to wall details that can help to ensure good seismic wall performance, the lessons learned from the 2010 Maule Chile earthquake provided previously herein can be used directly, as well as used in principle to apply to details used in current proprietary wall systems to address, for example, wall corners and vertical joints in the wall facing. Hence, the AASHTO LRFD Bridge Design Specifications (AASHTO 2012), specifically articles 11.6.5.6 and 11.10.7.4, now include recommendations for wall design and construction details that will help to improve wall seismic performance.

## Concluding Remarks

The 2010 Maule Chile earthquake provided a natural full scale laboratory to investigate the seismic performance of walls designed using the design code that has been used in US transportation infrastructure design, the AASHTO Standard Specifications for Highway Bridges (2002). Lessons learned from the wall performance observed in that earthquake, as well as wall performance observations from other earthquakes and recent seismic research, were useful for developing the first major update of the AASHTO seismic wall design specifications in almost 20 years. Overall, the updated wall design specifications are less conservative than what has been used in the past, improving the cost effectiveness of wall designs in seismically active areas. Yet, the addition of recommendations for improved wall design and construction details in the AASHTO specifications will help to obtain more consistent seismic performance of walls in the future.

Highlighted herein is the development background for some of the key changes recently implemented in the AASHTO LRFD Bridge Design Specifications. The improvements made to the estimation of seismic earth pressure are a good first step. However, it is recommended that the Koseki, et al. (1998) improvements to MO analysis be considered for future AASHTO specification improvements. There are other significant changes to those design specifications that could not be discussed in detail due to space limitations, such as, the use of soil cohesion in seismic design situations, and an improved internal seismic stability design procedure for reinforced soils walls. The AASHTO specifications, Section 11 and Appendix A11, (AASHTO 2012) should be consulted for additional background on these changes.

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