Stabilization of Alberta Highway Landslides using Pile Walls

Tarek S. Abdelaziz, Donald W. Proudfoot
Thurber Engineering Ltd., Edmonton, Alberta, Canada
Roger Skirrow
Alberta Transportation, Edmonton, Alberta, Canada

2011 Pan-Am CGS Geotechnical Conference

ABSTRACT
More than 25 pile walls have been constructed to remediate landslide movements affecting Alberta highways over the last three decades. Cantilever and tied-back pile walls have been used at numerous sites in Alberta. The pile types have varied from driven steel to cast-in-place concrete piles, while the tie backs have consisted predominantly of grouted and screw anchors. In some cases, composite pile walls consisting of cast-in-place below ground concrete piles and above ground H-piles with timber lagging were used. The paper summarizes pertinent details from 28 pile wall sites, and reviews the wall design details from several projects. These details are used to establish rules of thumb for when pile walls could be considered to remediate landslides, what types of piles could be used, and whether cantilever or tied back pile retaining systems are likely to be more cost-effective. Some instrumentation monitoring results such as wall deflections and anchor loads are also presented and discussed to demonstrate the performance of these retaining structures.

RÉSUMÉ
Au cours des trois denrnières décennies en Alberta, au delà de 25 murs de soutenement à membrures verticales ont été construits le long des autoroutes afin d'atténuer l'effet du mouvement causé des glissements de terrain. Des murs en porte-à-faux et à pieux avec parois ancrées ont été utilisés à plusieurs reprises sur ces sites en Alberta. Le type de pieu utilisé variait de pieux battux en acier à des pieux forés avec béton coulé-en-place, alors que le type d'ancrage utilisé fut majoritarement du type scellé par injection de coulis ou des ancrages visés. Dans certains cas, des murs composites ont été utilisés comprenant des pieux en béton coulé-en-place sous la surface du sol et des profilés d'acier en parois berlinoises avec blindage en bois au-dessus du sol. Cet article résume les détails pertinents de 28 sites ayant des murs de soutenement à membrures verticales composées de pieux, ainsi qu'une révision des details de la conception des murs de plusieurs projets. Ces détails permettent d'établir les règles-de-l'art utilisées afin déterminer dans quelles circonstances des pieux pourront être utilisés pour remédier un glissment, le type de pieu préférable, anisi que si un mur de soutement en porte-à-faux ou à parois ancrées est plus économique. Quelques données prises lors de la lecture d'instruments, dont la déflection des murs et les charges portées par les ancrages, sont aussi présentées et discutées afin de mettre en évidence la performance de ces ouvrages de soutenement.

1 INTRODUCTION

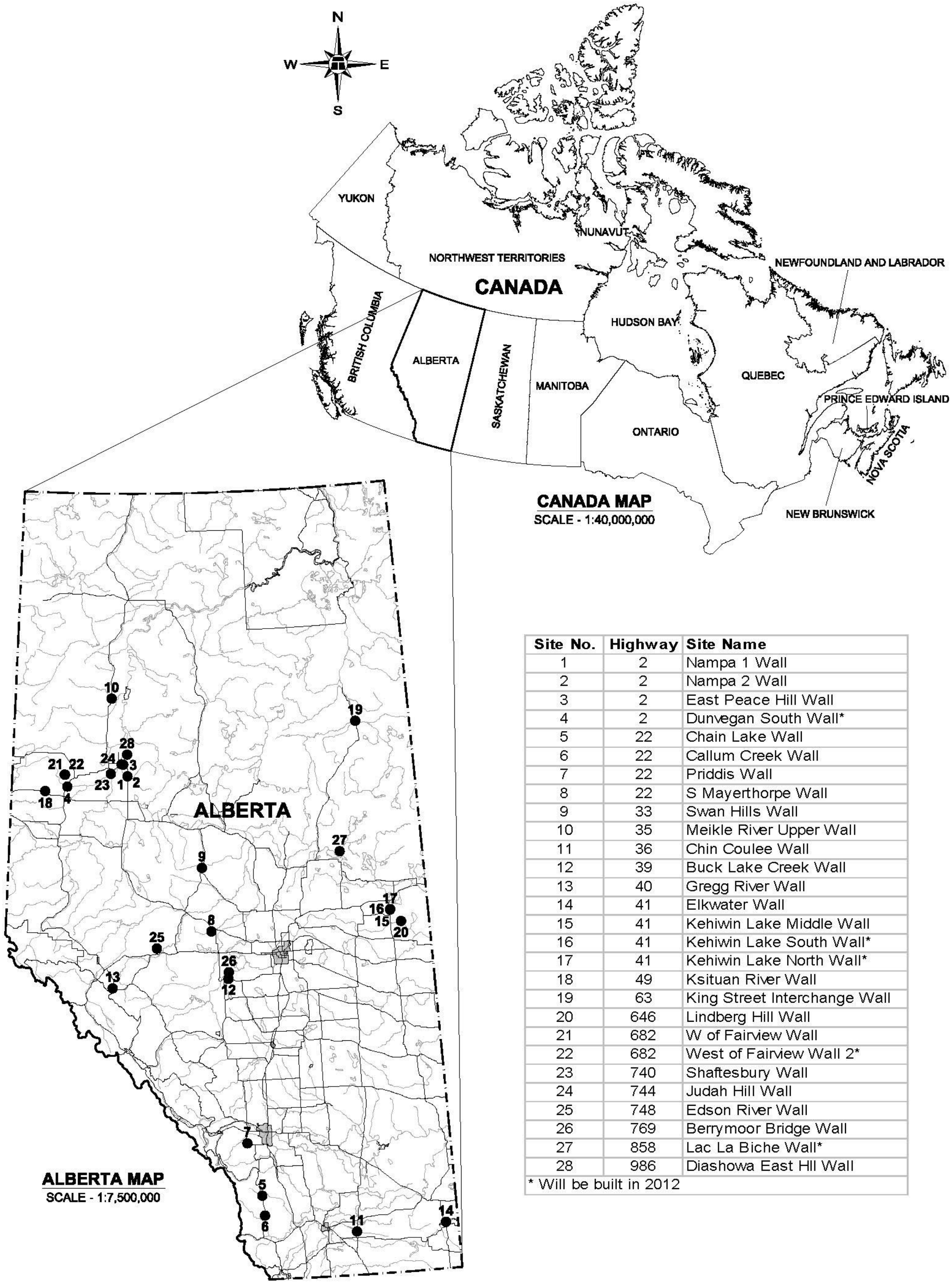
Landslides are a common geo-hazard that have affected highways in *Alberta* for many decades. More than 200 landslides affect Alberta highways, with a financial liability much greater than the budget allowance available to repair all the landslide problems. Consequently, Alberta Transportation (TRANS) initiated a "Geohazard Risk Management Program" to aide in prioritizing the repair work in a rational and defensible manner. The program includes annual site inspection visits and semi-annual instrumentation monitoring to assess the risk (probability x consequence) of failure at each geo-hazard site. Sites with high risk of failure have been remediated to maintain safety of highway travellers and integrity of existing infrastructure.

Site specific constraints such as conflicts with utility line, pipelines and existing infrastructure; requirements of approvals from regulatory authorities; minimal disruption of highway traffic; land acquisition issues; and existence of water bodies within or at the toe of the slide mass made the construction of pile walls a very attractive remedial measure. Over the last three decades, more than 25 cantilever and tied-back pile walls were constructed by TRANS to remediate landslides. The pile types have varied from driven steel to cast-in-place concrete piles, while the tie backs have consisted predominantly of grouted and screw anchors.

This paper reviews available data from 28 pile wall sites in *Alberta* to summarize existing information and provide suggestions for selecting the type of a pile wall and associated construction costs for preliminary assessments. Selected instrumentation monitoring results are also provided to highlight the importance of instrumentation in pile wall projects. A site location map showing pile wall sites is presented in Figure 1.

2 BACKGROUND

Numerous articles have been published with regard to the topic of slope stabilization using pile walls (e.g. Ito and Matsui (1975), Viggiani (1981) , Poulos (1995), Lee et al. (1995), Hassiotis et el. (1997), Chen and Martin (2002), Jeong et al. (2003), Cai and Ugai (2003), and Martin and Chen (2005). A brief summary of literature is provided herein with more attention to arching and group effects for one row of slope reinforcing piles (i.e. passive



Site No.	Highway	Site Name
1	2	Nampa 1 Wall
2	2	Nampa 2 Wall
3	2	East Peace Hill Wall
4	2	Dunvegan South Wall*
5	22	Chain Lake Wall
6	22	Callum Creek Wall
7	22	Priddis Wall
8	22	S Mayerthorpe Wall
9	33	Swan Hills Wall
10	35	Meikle River Upper Wall
11	36	Chin Coulee Wall
12	39	Buck Lake Creek Wall
13	40	Gregg River Wall
14	41	Elkwater Wall
15	41	Kehiwin Lake Middle Wall
16	41	Kehiwin Lake South Wall*
17	41	Kehiwin Lake North Wall*
18	49	Ksituan River Wall
19	63	King Street Interchange Wall
20	646	Lindberg Hill Wall
21	682	W of Fairview Wall
22	682	West of Fairview Wall 2*
23	740	Shaftesbury Wall
24	744	Judah Hill Wall
25	748	Edson River Wall
26	769	Berrymoor Bridge Wall
27	858	Lac La Biche Wall*
28	986	Diashowa East Hll Wall

* Will be built in 2012

Figure 1. Site location map showing pile wall sites in Alberta

piles). According to the available literature, piles are classified into active and passive piles. Active piles are subjected to a horizontal load at the head from superstructures and transmitting this load to the soil. Passive piles are more complicated because lateral forces acting on the piles are mobilized due to lateral soil movement. The movement of soil is a basic requirement to transfer lateral forces to passive piles through the arching effect. In general, arching is defined as the transfer of stress from yield parts of a soil mass (for landslides, it is the soil mass behind the pile wall) to adjoining less-yielding or restrained parts of soil mass (i.e. piles which constitute stiffer elements in a moving slide mass). For a single row of piles, arching usually develops for center-to center pile spacing (S) between 2 and 4 pile diameter (D). It has been shown that the arching between the piles can develop as long as the ratio of inter-pile ground displacement to the displacement of the pile head is maintained between 1 and 2 (at most). For pile spacing greater than 4D, piles act independently and soils tend to flow between the piles.

Although piles with closer spacing provide more confidence in the development of the arching effect, closer spacing piles provide less capacity due to the group effect. For a single row of piles, the group effect becomes insignificant for S/D values greater than or equal to 4. For example, a contiguous pile wall with S/D of 1 could exhibit 50 % of the lateral resistance of a pile wall with S/D of 4.

For pile walls, the embedment length of the piles below the slip surface dictates the behaviour of the wall. Small pile embedment into stable stratum below the slip surface is dominated by rigid-body rotation without substantial flexural distortion to engage the piles. It was found out that the critical embedment depth below the slip surface to achieve fixity condition at the base of the pile can range from 0.7 to 1.5 (depending on the strength of the stable stratum below the slip surface) the slide depth at the pile wall location.

3 PILE WALL DATABASE IN ALBERTA

3.1 Overview

Table 1 lists pertinent information regarding twenty-eight pile walls used to stabilize active landslides in *Alberta*. Some of these sites were constructed a long time ago and complete records could not be located in the archived documents. However, available information was collected and summarized in the table for completeness. For each of the sites, the table presents the slide depth at the pile wall location, type of piles and anchors, number and diameter of piles, total length of pile wall, center-to-center pile spacing, and construction costs. The construction costs are for the entire project and have not been normalized to 2010 rates. The table also includes spacing (S) to pile diameter (D) ratio (i.e. S/D) and embedment depth below slide (L_b) to slide depth (H_u) ratio (i.e. L_b/H_u).

The existing database indicated the presence of at least eighteen cantilever and ten tied-back pile walls in *Alberta*. All of the pile walls consisted of one row of piles. Six sites used below-ground driven steel piles, twenty-one sites used cast-in-place concrete piles, and one site, the Chin Coulee landslide, used soil nails as a below-ground reinforcement element to support the guardrail that was coincident with the crest of a large landslide. For the tied-back pile walls, the tie-back element primarily consisted of grouted bar anchors (7 sites) and screw anchors (2 sites). We had no information with regard to the tie-back elements used for the Nampa 1 site. In some of the pile walls, a cap beam, waler, or timber lagging was used to further restrain lateral movement, retain new fill placed to restore distressed crests of slopes, and/or distribute the anchor forces along the pile walls. In the Edson River site, the tops of the piles were connected to a gabion wall. In the Chin Coulee wall site, the vertically installed soil nails were connected at the top to a cast grade beam, on which a Mechanically Stabilized Earth (MSE) wall was constructed to retain the fill behind the wall.

3.2 Conventional Design Approach

The following provides a brief summary of common practice for the design of pile walls.

The pile wall design procedure includes the following steps: (a) simulation of existing slope failure (i.e. a slope with a Factor of Safety (FOS) of 1.0); (b) calculation of the pile wall force required to increase the long-term FOS to 1.3; (c) development of geotechnical design parameters including lateral soil model of the wall, adhesive values for bond zone of anchors (if any), proposed location of the wall, allowable deformation at the pile head, and group reduction factors; and (d) structural design of pile wall.

In Step (b), the location of the pile wall is usually selected to optimize the design. In the majority of Alberta pile walls, piles were located within 3 m from the edge of the highway shoulder for the following reasons: (i) minimizing the probability of new local slides toeing above the pile wall location; (ii) reducing the volume of supported slide mass above the pile wall to reduce the required stabilizing force (F_s); and (iii) to reflect constructability considerations. The magnitude of the stabilizing force was found to be highly dependent on whether the passive resistance of the downslope soil block in contact with the pile wall above the slip surface (H_u) should be discounted in the design. A long-term FOS of 1.3 is usually considered where continued highway maintenance could be carried out until the pile wall mobilizes the full stabilizing force. A tolerable lateral deformation of 0.5-1% of the wall height (H) or 70 mm of pile head deflection, whichever is less, is typically specified in pile walls for TRANS highway projects.

In Step (d), the piles are designed to have sufficient embedment (L_b) into the stable soils below the slip surface for a selected pile wall configuration, the structural engineer undertakes two separate verifications: (i) Serviceability Limit State (SLS) verification by using un-factored (working or actual) geotechnical loads and resistance of soil and pile elements to ensure that the calculated pile wall deflections are less than or equal to the specified deflection threshold and (ii) Ultimate Limit State (ULS) verification to simulate a collapse mechanism of the structure and calculate the maximum bending moments experienced by the piles. For the SLS

Table 1. Summary of pile wall details in Alberta

Site No.	Site Name	GRMP No	Highway	Slide Plane Depth, Hu (m)	Year Installed	Type of Wall	Reason	# of Piles (# Rows)	Length of Wall (m)	Installed Depth, H (m)	Pile Section	Reinf.	Center- to Center Pile Spacing, S (m)	Waler/ Cap Beam	Tie back	Lb/Hu	S/D	Construction Cost	Cost/ m of wall
1	Nampa 1 Wall	PH13	2:60	2-4	1996	CIP piles	Topography, creek at toe	18(1)	34	12	CIP 610 mm dia	310 x 79 H-pile	2	CIP Beam	na	3.0	3.3	nr	nr
2	Nampa 2 Wall	PH13	2:60	2-4	1991	CIP piles	Topography, creek at toe	16(1)	30	12	CIP 610 mm dia	310 x 79 H-pile	2	Timber Lagging	na	3.0	3.3	nr	nr
3	East Peace Hill	PH02-10	2:60	nr	nr	CIP Piles	Topography, deep creeping valley	nr	nr	nr	nr	nr	nr	CIP Beam	na	nr	nr	nr	nr
4	Dunvegan South Wall	PH35	2:68	6.5	2011	CIP Piles	Topography, deep creeping valley	63(1)	76	15.5	CIP 1200 mm dia	Steel Bar, riser of HP pile	1.2	CIP Beam	Grouted 26 mm dia double corrosion protected bars	1.4	1	$ 2,380,000	$ 31,315
5	Chain Lake Wall	S16	22:08	6	2006	Driven H-Pile	Topography, culvert issue	59(1)	70	14	Driven H310x110	na	1.2	Timber Lagging, W310x79 Waler	Grouted Dwidag	1.3	3.8	$ 1,634,000	$ 23,340
6	Callum Creek Wall	22-8-1	22:08	8	2005	Driven H-Pile	Creek at toe plus adverse surface water and groundwater conditions.	31(1)	60	15	Driven H310x110	na	2	Timber Lagging, W310x107 Waler	Grouted 32 mm Dwidag	0.9	6.3	$ 892,000	$ 14,867
7	Priddis Wall	S02	22:14	7.3-10.3	1992	CIP Piles	High GWT, topography	31(1)	70	16	CIP, Upper 5 m @ 1067 mm dia, lower 11 m @ 760 mm dia	Steel Bar	2-3	No	na	1.2	1.8-3.9	na	nr
8	S. Mayerthorpe Wall	NC63	22:32	3	2008	Driven H-Pile	Drainage course at toe, lowest cost option	42(1)	25.4	12	Driven H310x94	na	0.62	No	na	3.0	2.1	$ 163,000	$ 6,500
9	Swan Hills Wall	SH01	33:12	6	1989	CIP Piles	Topography, culvert issue	82(1)	82	6-12	CIP 1067 mm dia	Steel Bar	1	No	Grouted 36 mm dia. double corrosion protected bars	1.0	0.9	na	nr
10	Meikle River Upper Wall [a]	PH45	35:08	10	1997	CIP Piles	topography, long creeping valley	77(1)	110	22- 24	CIP 760 mm dia	H-Pile	1.5	Yes CIP beam in 1998	na	1.2-1.4	2	$ 126,000	$ 1,146
11	Chin Coulee Wall	S5	36:02	2	2008	Soil Nail/MSE	Practical to only stabilize crest of slide	10 (1)	10	6	50 mm dia hollow shot rod	na	1	CIP footing to support MSE wall	na	2.0	na	na	nr
12	Buck Lake Creek Wall	NC19	39:06	3-3.5	2003	CIP Piles	Topography and dip of slide plane	36 (1)	70	10	CIP1200 mm dia	Steel Bar	2	No	na	1.8-2.3	1.7	$ 304,000	$ 4,342
13	Gregg River Wall	NC50	40:28	6	2010	CIP Piles	Topography, river at toe	30 (1)	82	18	CIP1800 mm dia	Steel Bar	2.8	No	na	2.0	1.6	$ 1,056,000	$ 12,875
14	Elkwater Wall	S26	41:03	2-4	2011	Driven steel H-Pile	Minimize traffic distrubtion, reduced affects on park land	43 (1)	50	10	HP360x174	na	1.22	Steel Waler	na	1.5-4	3.3	$ 660,000	$ 13,200
15	Kehiwin Lake M. Wall	NC24	41:23	5.8	2009	CIP Piles	Adjacent lake	41 (1)	144	15	CIP 1800 mm dia	Steel Bar	3.6	No	na	1.6	2.2	$ 1,088,000	$ 7,555

Table 1 (Continued)

Site No.	Site Name	GRMP No	Highway	Slide Plane Depth, Hu (m)	Year Installed	Type of Wall	Reason	# of Piles (# Rows)	Length of Wall (m)	Installed Depth, H (m)	Pile Section	Reinf.	Center to Center Pile Spacing, S (m)	Waler/ Cap Beam	Tie back	Lb/Hu	S/D	Construction Cost	Cost/ m of wall
16	Kehiwin Lake S. Wall	NC24A	41:23	8 to 9		CIP Piles	Adjacent lake	69 (1)	126	15	CIP 1200 mm dia	Steel Bar	1.8	CIP Waler	Grouted 36 mm dia. double corrosion protected bars	0.7-0.9	1.5	$ 2,338,869	$ 18,562
17	Kehiwin Lake N.Wall	NC24B	41:23	4	2011	CIP Piles	Adjacent lake	29 (1)	113	10	CIP 1500 mm dia	Steel Bar	4	No	na	1.5	2.7	$ 700,000	$ 6,195
18	Ksituan River Wall	GP12B	49:04	5	2003	CIP piles	Adjacent pipeline	60 (1)	111	20	CIP 1067 mm dia	Steel Bar	1.8	No	na	3.0	1.7	$ 1,015,000	$ 9,144
19	King Street Interchange Wall	NC35	63:11	5	2001	CIP Piles	Overpass headslope creeping at intolerated rates by bridge structure	30 (1)	35	10.5 (750 mm piles) to 11.5 (1200 mm piles)	CIP 1200 mm dia along the toe of head slope and CIP 750 mm dia. long the sides of the headslope	Steel Bar	2.4 - 3	No	na	1.3-1.6	2-4	$ 250,000	$ 7,143
20	Lindberg Hill Wall	NC25	646:04	4 to 8	2007	CIP piles	CNRL gas wells at bottom of slope	46 (1)	85	14 to 20	CIP 1200 mm dia	Steel Bar	1.8	CIP Waler	Grouted 36 mm dia double corrosion protected bars	1.4-2.5	1.2	$ 1,990,420	$ 23,416
21	W of Fairview Wall	PH4	682:02	6	2008	CIP Piles	Slide geometry, topography	67 (1)	102	14	CIP 1067 mm dia	Steel Bar	1.5	No	na	1.3	1.4	$ 1,585,000	$ 15,539
22	West of Fairview Wall 2	PH27	682:02	5	2011	CIP Piles	Combination solution ties to minor road realignment	34 (1)	60	18	CIP 1500 mm dia	Steel Bar	1.8	No	na	2.6	1.2	$ 850,000	$ 14,167
23	Shaftesbury Wall	GP31	740:02	7	2009	Driven H-Pile	Slide geometry, topography	114 (1)	85	13 m outside of slide, 20 m within slide	HP310 x 79	na	0.75	Walers	50 mm square section, helical anchor, 4 helixes, installed 22 to 32 m deep	1.9	2.5	$ 1,856,000	$ 21,835
24	Judah Hill wall [b]	PH33	744:04	20	1988 to 1989	CIP Piles	Proactive measure to prevent headscarp retrogression	30 (1)	nr	20	CIP 760 mm dia	Steel Bar	1.5	No	Grouted anchors were used in 1994 to hold back the wall after being	nr	2	nr	nr
25	Edson River Wall	NC15	748:02	7	1999	CIP Piles	River at toe, minimal traffic disruption	65 (1)	80	20	CIP 600 mm dia	H-Pile	2	Gabion Wall	na	1.9	3.3	$ 448,000	$ 5,600
26	Berrymoor Bridge Wall	NC07	759:02	6.5 to 8	2004	CIP Piles	Topography	45 (1)	90	12- 14	CIP 1220 mm dia	Steel Bar	2	CIP Beam	Helical Anchor, 4 helixes	1.3-2.4	1.6	$ 1,145,000	$ 12,722
27	Lac La Biche Wall	NC58	858:02	5.5	2011	Driven Piles	Land issues, Utility line , high GWT	146 (1)	90	15	HP 310x110	H-Pile	0.62	No	na	1.7	2	$ 800,000	$ 8,888
28	Diashowa East Hill Wall	PH43	986:01	6.5	2004	CIP Piles	Topography, creek at toe	53(1)	117	19.4	CIP 1524mm dia.	Steel Bar	2.3	CSP Beam	na	2.0	1.5	na	nr

(a) Short screw anchors were installed below highway lane surface in 1996 to reinforce upper portion of slide mass. Due to additional movement, Meikle River piles were constructed in 1998 , but not that deep to completely stabilize the landslide.

(b) Judah Hill piles were installed immediately above slip surface as a precautionay measure to protect highway from headscarp retrogression. River toe erosion occurred, causing movement of soil downslope of the pile wall. Pile wall was exposed and grouted anchors were installed to further enhance the stability of the wall.

na = not applicable; nr= no records available verification, the structural consultant typically uses the stabilizing force corresponding to a FOS of 1.3 to calculate the pile wall deflection, which is expected to result in a somewhat conservative design.

3.3 Pile Spacing and Embedment Depth

Figure 2 shows the distribution of S/D for all of the pile wall sites. The ratio typically varied between 1 for tangent piles and 4 for spaced piles, with average values of 1.5 and 2 for cantilever and tied-back walls, respectively. Figure 3 shows the distribution of L_b/H_u for all of the pile wall sites. The ratio varied from 0.7 to 4, with average values of 1 and 1.5 for cantilever and tied-back walls, respectively.

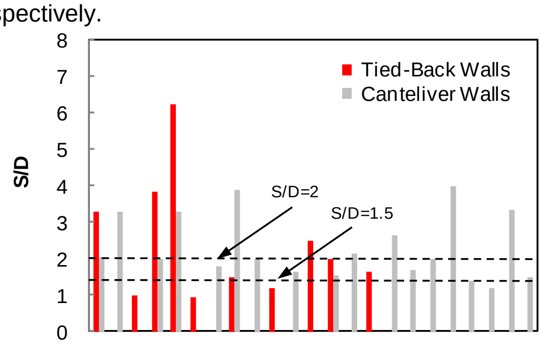


Figure 2. Distribution of S/D for pile walls

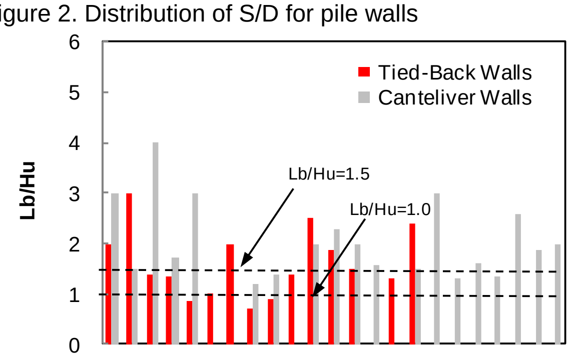


Figure 3. Distribution of L_b/H_u for pile walls

3.4 Instrumentation Monitoring of Pile Walls

Instrumentation is used to monitor long-term performance of pile walls, confirm design assumptions, and modify construction operations. Slope inclinometers, installed inside cast-in-place concrete piles, have been used to monitor the lateral deflection of the piles. Vibrating wire load cells have also been used to measure the anchor loads.

Figure 4 presents an example of slope inclinometer plots for a cantilever pile wall. Plots of incremental and cumulative deflections from one of the slope inclinometers installed at Site 10, Meikle River upper wall, are shown in this figure. It should be noted that positive and negative deflections indicate downslope and upslope movements, respectively. The incremental deflection plot shows no distinct kink in the plot at the slip surface (10-12 m depth). This indicates that the pile wall successfully retained the slide mass without sacrificing the integrity of the wall. The cumulative plot indicated that the pile head has deflected downslope of the highway by about 100

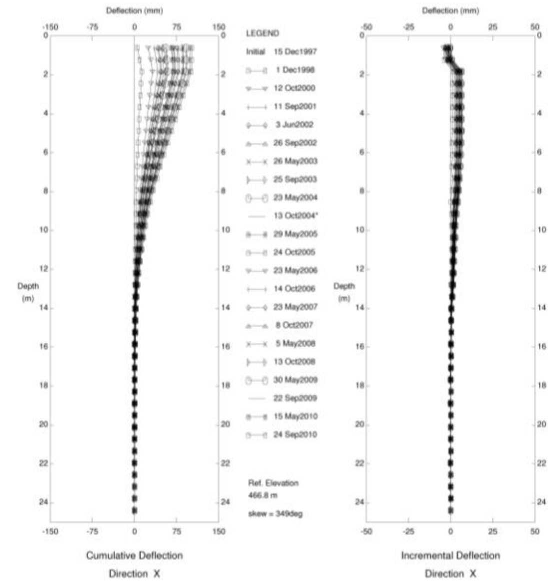


Figure 4. Incremental and cumulative deflection versus depth plots for the Meikle River upper wall.

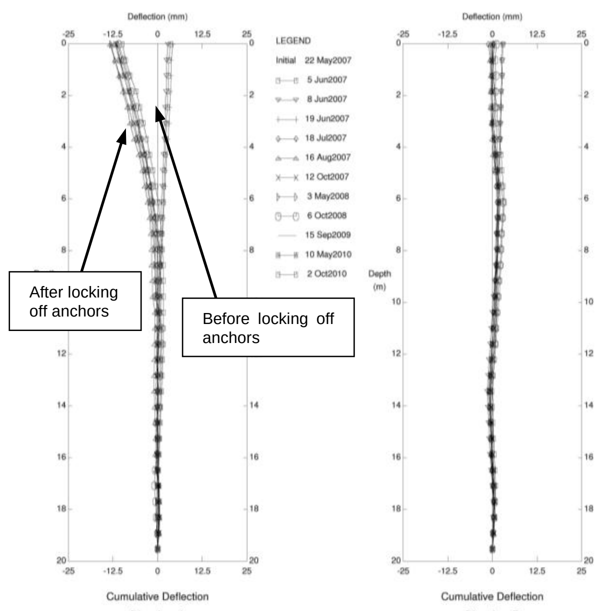


Figure 5. Incremental and cumulative deflection versus depth plots for the Lindbergh Hill wall

mm between December 1997 (installation date) and September 2010 (i.e. wall moved at a rate of 6.6 mm/year) and that the lateral deflection occurred primarily between the top of the wall and 14 m depth (2-4 m below the location of the slip surface). The plots basically pointed out that the pile wall was embedded sufficiently into the stable stratum below the slip surface to develop the required resistance and stabilize the slide movement, and deflected by about 0.4 % of its height.

Figure 5 presents an example of the slope inclinometer plots for a tied-back pile wall. Plots of incremental and cumulative deflections from one of the slope inclinometers installed at Site 20, Lindbergh Hill wall, are shown in this figure. At this site, the grouted anchors were locked-off one month after the installation of piles was completed. It can be seen from the cumulative deflection plot that the wall deflected a bit in the downslope direction prior to locking off the anchors (i.e. cantilever type pile wall behaviour). Immediately after locking-off the anchors, the pre-stress forces in the anchors pulled the wall into the upslope direction (towards the highway) by about 12.5 mm. Minimal movements were noted in the slope inclinometer after applying the pre-stress forces in the anchors. In this case, the slope inclinometers were very helpful during and after construction to capture slide movements prior to the locking off the anchors and monitor the effect of locking-off anchors on the overall behaviour of the wall.

Table 2 summarizes the lateral pile head deflection (δ) to pile depth (H) ratio from available slope inclinometer data. It can be seen from the table that cantilever pile walls deflected by 0.1 to 0.5%, with a typical value of less than 0.5 %, of the full height of the wall. The available information for tied-back walls indicated that the pile walls deflected by less than 0.05 % the full height of the wall. Tied-back walls exhibited less deflection compared to cantilever walls due to the pre-stress forces in the anchors. In TRANS projects, a 70 mm pile head deflection was considered to be a reasonable upper threshold for deflection. For the Meikle River slide, one of the piles has deflected by more than 70 mm. The ongoing monitoring will determine whether supplementary measures are required to fully restrain the landslide block at this site.

Table 2. Summary of δ/H at some pile wall sites

Site Name	Instrumentation Period	Wall Type	δ (mm)	δ/H (%)
Meikle River	Dec. 97 – Sep 10	CIP- C [1]	58-117	0.3-0.5%
Kehiwin Lake M. Wall	Mar. 09 – Oct. 10	CIP- C[1]	4-5.6	0.2%
King Street Interchange	Feb. 02 – Oct. 07	CIP-C[1]	15	0.1%
Lindbergh Hill	May 07- Oct.10	CIP-T[1]	2-6	0.04%
W. of Fairview	Jan. 08- Sep. 10	CIP- C[1]	35-45	0.3-0.4%
Berrymoor Bridge	Mar. 04- Oct. 07	CIP-T[1]	0.7-1	0.05%
Daishawa East Hill	Oct. 04- Sep. 10	CIP- C[1]	20-37	0.1-0.2%

[1] CIP,T, and C denote cast-in-place, tied-back and cantilever wall, respectively

Figure 6 shows the variation of anchor loads with time for the Lindbergh Hill pile wall. Although the anchor design loads dropped by 15 to 30 % one month after locking off the loads in 2007, the variation in loads afterwards remained negligible in the majority of monitored anchors and varied by 6 to 9%. The reduction in the anchor loads was probably due to the non-simultaneous locking of anchors and re-distribution of loads along the wall. No evidence of cracking other than reflective cracks was noted on highway surface and the site continued to perform satisfactorily.

4 SUGGESTIONS FOR SELECTION OF PILE WALL TYPE, ASSOCIATED BALL PARK COSTS, AND MAINTENANCE ISSUES

Site constraints or slide geometry may render the pile wall remedial measure as the preferred remedial option. Some of the reasons underlying the selection of a pile wall are listed below.

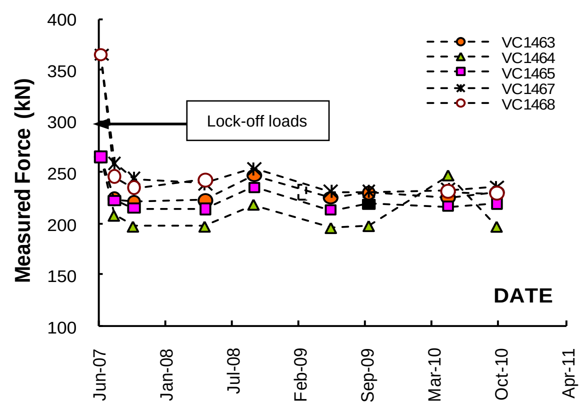


Figure 6. Variation of anchor loads with time for the Lindbergh Hill wall

1- Presence of a large landslide with its headscarp along the highway surface or immediately downslope of the highway surface.
2- Conflict between other remedial measures and location of existing utility lines, pipeline and infrastructure.
3- Construction delays due to requirements of approvals from regulatory authorities and land acquisition issues.
4- Existence of water bodies or environmentally sensitive areas at the toe of the slide mass.
5- Minimal disruption to highway traffic.

Based on the available information for walls constructed after 2005, Table 3 provides suggestions for the selection of pile wall type and associated costs for preliminary planning of pile wall construction projects. As the table shows, the construction cost of a tied-back wall is approximately double that of a cantilever pile wall, which makes tied-back walls not cost effective for shallow slide repair projects but more appropriate when slip surface becomes deeper than 6 m.

Table 3. Suggestions for selection of pile wall types and associated ball park construction cost.

Slip surface depth at proposed pile wall location (m)	Pile wall type	Cost/m of wall
0 to 3	Cantilever pile wall (H Piles)	\$6,500-\$7,500
4 to 6	Cantilever pile wall (CIP Piles)	\$12,000-\$16,000
Greater than 6	Tied-back pile wall	\$20,000-30,000

For shallow slides up to 3 m, it was found out that driven steel piles are the preferred pile elements due to quick installation rates. For example, although dependent on the size of pile rig , it may take 1 day to install 8- 12 m deep H 310x110 steel piles as compared to 2 days to install 3- 12 m deep 1200 mm diameter CIP piles. It should be noted however that the construction of driven steel piles may impose construction constraints due to induced vibrations. It was typically suggested in previous projects to install steel piles every third pile to allow time for partial pore-water dissipation. Vibration and noise may be more of an issue in urban areas, but don't typically affect TRANS projects which are normally in rural settings.

For preliminary design purpose, and based on available information from pile wall sites, Eq. 1 may be used to estimate the stabilizing force corresponding to a FOS of 1.3.

$$Fs = ((\gamma Hu^2)/2)) x\ k_{ls} \quad [1]$$

where Fs is the stabilizing force, γ is the bulk unit weight of soil, Hu is the depth is slip surface below the top of the wall; and k_{ls} is a landslide-related lateral pressure coefficient (a value of 0.7 may be used for preliminary assessment).

If a pile wall option is selected, highway maintenance should be expected until the pile wall mobilizes the required force to retain the landslide mass. Typically, it takes up to 3 to 4 years for cantilever pile walls to mobilize the stabilizing force. However, in the case of tied-back walls the stabilizing force is locked in during construction with minimal movement following construction unless there is relaxation or creep in the anchors.

In some pile wall sites, it occurred that the slide mass located downslope of the wall moved, creating a gap and a sharp drop along the face of the wall and accordingly safety hazard for runaway vehicles. In this case, the installation of a guardrail along the top of piles or cap beams is considered to be a potential cost-effective solution to eliminate such hazard. All of the above factors should be considered when a decision is made of whether or not to use a pile wall.

5 SUMMARY AND CONCULSIONS

This paper provided a synopsis of existing information regarding 28 pile walls constructed in *Alberta* to stabilize active landslides that were affecting provincial highway. The summarized information included depth of slip surface, type of piles and anchors, size and spacing between piles, depth and length of pile wall and associated construction costs. Examples of instrumented pile wall sites were also presented. Suggestions were established from existing information to provide practitioners with a tool for preliminary selection of pile wall types and determination of associated ball park construction costs.

From the existing information, the following conclusions are drawn:

1- Pile walls are an effective means to stabilize highways affected by landslides in *Alberta*.

2- Pile walls in *Alberta* have traditionally been designed with allowable deflections of 0.05% and 0.5 % of the full wall height or 70 mm, whichever is less, for tied-back and cantilever pile walls, respectively.

3- Cantilever pile walls could be considered to stabilize up to 6 m deep landslides. Tied-back walls with single or multiple levels of anchors should be considered for landslides deeper than 6 m. The construction cost of a tied-back wall is approximately double that of a cantilever pile wall.

4- For preliminary design purposes, a lateral earth pressure coefficient of 0.7 may be used to estimate the stabilizing force corresponding to a FOS of 1.3.

5- Instrumentation monitoring is a viable method to confirm long-term performance of pile walls, confirm design assumptions, and assess the impact of construction activities on active landslides.

ACKNOWLEDGEMENTS

The authors would like to acknowledge TRANS for providing data to complete this paper, and TRANS' staff and Consultants participated in the repair and monitoring of pile wall landslide sites, including Thurber Engineering Ltd., EBA Engineering Consultants, AMEC Earth & Environmental, Karl Engineering Consultants Ltd., GAEA Consulting Ltd, and Klohn Crippen Berger Ltd.

REFERENCES

Ito, T.S., and Matsui, T. 1975. Methods of estimate lateral force acting on stabilizing piles. *Soils and Foundations*, 15(4): 43 - 60

Poulos, H.G. 1995. Design of reinforcing piles to increase slope stability, *Canadian Geotechnical Journal,* 32: 808-818

Viggiani, C. 1981. Ultimate lateral load on piles used to stabilize landslides, *Proc. 10th Int. Conf. Soil Mech. and Foundation Eng.*, Stockholm, 3:555-560

Lee, C.Y., Hull, T.S., and Poulos, H.G. 1995. Simplified pile-slope stability analysis, *Computers and Geotechnics*, 17:1-6

Hassiotis, S., Chameau, J.L., and Guaratne, M. 1997. *Journal of Geotechnical and Geo-environmental Eng.*, 123(4): 314-323

Chen, C.-Y., Martin, G.R. 2002. Soil-structure interaction for landslide stabilizing piles. *Computers and Geotechnics*, 29: 363-386

Jeong, S., Kim, B., Won, J., and Lee, J. 2003. Uncoupled analysis of stabilizing piles in weathered slopes. *Computers and Geotechnics*, 30: 671-682

Cai, F., Ugai, K. 2003. Response of flexible piles under laterally linear movement of the sliding layer in landslides. *Canadian Geotechnical Journal*, 40: 46-53

Martin, G.R., Chen, C.-Y. 2005. Response of pile due to lateral slope movement. *Computers and Geotechnics*, 83: 588-598