# STEEL PLATE

# SHEAR WALLS

## *Now Performing on the Main Stage*

*Robert G. Driver, P.Eng. and Gilbert Y. Grondin, P.Eng.*

### A new player in a leading role

For many years, there has been a player "waiting in the wings"—a player whose performance has been under-appreciated in spite of consistently good reviews. Once considered simply a lateral load-resisting system with great potential, the steel plate shear wall concept is now rapidly gaining popularity with owners and consultants for consideration in building construction projects. The system is being recognized for its benefits in both new and rehabilitation work primarily in seismically active zones, although its high elastic stiffness is also desirable for resisting wind forces. Steel plate shear walls consist of steel plates one story high and one bay wide installed vertically within a building frame and connected to the surrounding beams and columns, a configuration which has been described as being analogous to a vertical cantilevered plate girder.

After thirty years of research and development, the technical merit of

Figure 1. Steel plate shear walls (SPSW): Canam Manac Head Office Expansion.

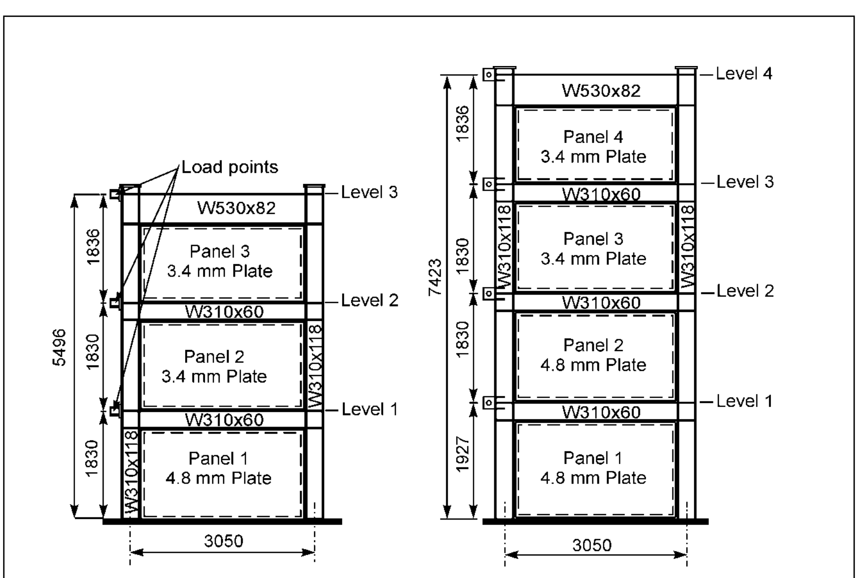



Figure 2. Large-scale steel plate shear wall test specimens.

the system is now difficult to deny. Although a multiplicity of forms have been studied, the particular configuration gradually emerging as providing the most desirable trade-off between technical performance and economics is the unstiffened thin-panel concept, pioneered by Kulak at the University of Alberta in the early 1980s. Since that time and although research at the University of Alberta has been ongoing, many other researchers in North America and elsewhere have made significant contributions to the evolution of the system as we know it today.

Recently it has also been demonstrated through a detailed study presented by Timler and Ventura (1999) that steel plate shear walls can provide significant cost savings to a project. These cost savings arise, in part, through minimizing construction time and impacting the cost of other elements in the building, such as the foundation. Other desirable aspects of steel plate shear walls include the ability to complete stairs and elevators rapidly during construction and the fact that these thin shear walls occupy little rentable floor space.

## From page to stage

One example of a modern steel structure that features steel plate shear walls is the recently completed head office expansion of the Canam Manac Group located in St.-Georges de Beauce, Quebec, a seismic zone 3. The six-story expansion adds over 3,700m$^2$ (39,830 sq. ft.) of office space to the building, plu s an extension to an underlying hotel. The project utilized 46 tons of structural steel and 161 tons of steel joists.

The optimal structural solution for resisting lateral loads in the north–south direction was determined to be a series of steel plate shear walls located around the elevator shafts, as shown in Figure 1. A moment frame alternative was found to be less cost-effective, due in part to the lack of repetition at the connections because of the relatively complex building configuration. The required flexibility of the architectural layout led to the selection of the elevator core location for the shear walls, limiting the shear wall width to approximately 2.6m (8.5′) center-to-center of the columns. The shear walls have an overall height of 23m (75′).

The steel plate shear wall infill plates are 4.8mm (.19″) thick, and the columns are W250s. At the floor levels, double C200x17 (equivalent US C8x11.5) members were used to anchor the tension field in the infill plates as well as carry gravity loads around the shaft. The shear walls were fabricated in two tiers to optimize column selection and facilitate erection. The tiers were bolted together with a slip-critical field splice at mid-height using A325 bolts and a single-sided 4.8mm (.19″) splice plate at the infill panel joint to ensure proper load transfer.

## From "improv" to script

Despite the acceleration of recent research efforts pertaining to steel plate shear wall behavior, and the increased number of progressive designers considering the concept for modern buildings, the system could not reach the mainstream of construction without codified guidelines. Design standards need to provide to designers methods that are technically sound, that identify important pitfalls, that are not beyond the reasonable capabilities of a typical design office and that are relatively transparent in their interpretation so that they can be applied by competent engineers to atypical situations. The Canadian standard for structural steel design, CAN/CSA-S16 (CSA. 2001), is the first standard to include provisions for unstiffened steel plate shear wall design. In 1994, a non-mandatory appendix "Design Requirements for Steel Plate Shear Walls" was included in the standard that provides both an analytical procedure and design guidelines. The presence of this appendix sparked productive discussions within the general structural engineering community.

The upcoming (2001) edition of CAN/CSA-S16 will see the non-mandatory steel plate shear wall appendix move into the mandatory main body of the standard, while incorporating many changes arising from our increased understanding of the behavior of the system. Since the publication of the 1994 standard, research by Driver et al. (1998) and others has provided substantial additional evidence as to the cyclic performance of multi-story steel plate shear walls when loaded to extreme deflections. Based on this research, it is anticipated that the inelastic seismic force reduction factor, *R*, for the system (including a moment-resisting boundary frame for redundancy) will be increased from its present value of 4, which is currently equivalent to that of ductile moment-resisting frames and eccentrically braced frames.

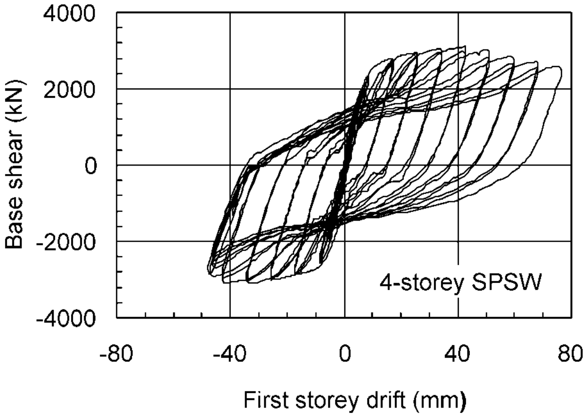
A fundamental addition to CAN/CSA-S16 is the explicit requirement that capacity design principles be adhered to for seismic design. To this end, design-overturning moments for steel plate shear walls are amplified so as to assure that the primary mode of energy dissipation in an earthquake is that of yielding of the infill plate. The demand on the supporting columns is thereby reduced. Additional anticipated modifications to the standard include the inclusion of a new clause outlining a minimum stiffness requirement for columns bounding a steel plate shear wall panel. The purpose of this clause is to assure that a relatively uniform tension field is developed in the infill plate and prevent significant "pull-in" of the columns. Many other refinements to the existing procedures will be included in the next edition of the standard to reflect the progress made through recent research efforts.

## Dress rehearsals—honing the performance

In an effort to provide experimental evidence of the performance of thin-panel steel plate shear walls, a series of two large-scale laboratory tests were conducted at the University of Alberta (Driver et al. 1998, 2001). The first test was on a four-story specimen that was 7.5m (25′) tall and 3.4m (11′) wide, as depicted in Figure 2a. The columns consisted of W310x118 members and the beams were typically W310x60, with the roof beam being a deeper W530x82 to anchor the tension field that develops in the infill plate. The infill plate thicknesses were 4.8mm (.19″) in the lower two storys and 3.4mm (.13″) in the upper two. The beams were connected to the columns with moment-resisting connections in order to investigate the beneficial effects of having a dual frame–shear wall system.

Gravity loads of a magnitude representing reasonable unfactored values for a typical building were applied to the columns in order to include P-delta effects in the test and horizontal loads were applied in fully reversed cycles at each story. Thirty cycles of load, including 20 inelastic cycles, were applied to the specimen up to a maximum story drift of 4%. Tears in the lowest infill plate began forming after about 22 cycles but were found to have little effect on the stiffness and shear carrying capacity of the wall due to the ability of the plate to effectively redistribute the load. Ultimately, the deformation of the most severely displaced story was more than nine times that at which the material began to show significant yielding, as can be seen in the base shear vs. interstory drift curve in Figure 3a. The specimen proved to be initially stiff, very ductile and it exhibited stable hysteresis behavior with significant energy dissipation. This test provides large-scale evidence of the superior cyclic behavior of steel plate shear walls.

Most of the damage observed in the first test was concentrated in the lowest story, and although the infill plate in the second story buckled and deformed plastically during the test, no significant damage was evident in the top three storys. For this reason, the lowest story was removed from the test specimen, and the top three storys were re-tested. For the second test, the overall height of the specimen was 5.5m (18′), as shown in Figure 2b. The main objectives of this test were to increase the database of test results on large-scale unstiffened steel plate shear walls, monitor closely the behavior of the frame members in the test speci-



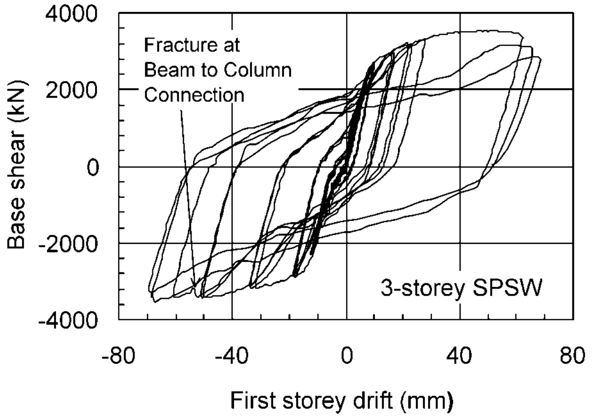




*Figure 3. Base shear versus first story drift. (left) Four story shear wall. (right) Three story shear wall.*

men and assess the residual capacity and performance of a wall that had previously been subjected to an earthquake. The base shear vs. interstory drift curve for the lowest story for the re-test are shown in Figure 3b. As can be seen in the figure, a displacement ductility ratio of about 9.5 was achieved in this story at a drift of approximately 3.7%.

As indicated in Figure 3b, at a base shear of 3400 kN (and after a total of 50 load cycles), sudden rupture of the beam flange to column flange connection took place at the top flange of one connection of the beam at level 1. This resulted in a small drop of shear force in the test specimen but, as shown in Figure 3b, despite the beam failure, the full load was quickly recovered and the cycle was completed under a base shear of 3420 kN. The overall effect of the connection fracture on the shear versus drift response is seen to be minimal, which is attributed to the redundancy of the system. Since one objective of the test was to evaluate the maximum capacity of the wall and investigate the behavior of infill plates under extreme loading conditions, the beam-to-column connection was repaired and the test resumed. In the final cycles, the wall was loaded to the stroke capacity of the hydraulic actuators in both directions.

The test specimen, shown in Figure 4 before and after testing, underwent a large amount of plastic deformation in the lower two storys. Between the two tests, a total of 54 cycles of load were applied to the test specimen, with 34 of these cycles exposing the specimen to significant yielding. This is clearly more severe than the number of inelastic cycles that a shear wall would be expected to resist during an earthquake. It can be seen that, despite the fact that panel 1 sustained the most damage, panel 2 also sustained a lot of plastic deformation, which indicates the potential for designing a shear wall so that energy is dissipated in multiple panels over the height of the building. Panel 3 also shows signs of extensive yielding, although the damage in that panel is much less severe than in the other two.

The area enclosed by the hysteresis curves is a measure of the energy dissipated by the system in resisting the particular load or displacement history. Figure 3 shows that the hysteresis loops generated are fairly wide, indicative of good energy absorption, although they are somewhat pinched because of buckling of the infill plates. Furthermore, in both tests the energy absorbed increased in each cycle throughout the duration of the test. Figure 5 shows a summary of the cumulative energy absorbed by each panel throughout the second test (subsequent to the 30 cycles from the first test). As expected, because the behavior was essentially elastic, little energy was absorbed in the first ten cycles of the test. By the end of the test, panel 1 had contributed 65% of the total energy dissipation, and panel 2 contributed 30%.

The test on the three-story steel plate shear wall confirmed once again the high initial stiffness, large energy dissipation capacity and great ductility of the system, even after a large number of extreme load cycles. It also demonstrated its redundancy, since fracture of one of the beam-to-column connections led to very little loss in load carrying capacity. Moreover, the test illustrated that steel plate shear walls can be designed to allow for large energy dissipation in several panels.

These two large-scale tests have demonstrated that steel plate shear walls exhibit robust cyclic performance under extreme loading conditions. The results have lead to the reevaluation of the force reduction factor, *R*, as described above.

## A change of venue

Although steel plate shear walls are, not surprisingly, associated primarily with steel buildings, interest is also emerging in rehabilitating older reinforced concrete frame buildings with steel plate infill panels. A recent project that used this rehabilitation technique is the three-story historic library in Salem, OR (Robinson and Ames 2000). In this project, the shear panels were connected to the existing concrete structure with drilled anchors.

Although there are exceptions, the use of steel plate shear walls for rehabilitation has not yet become common. The reason for this is undoubtedly the paucity of research information available specifically addressing rehabilitation issues. When rehabilitating steel frame buildings by incorporating steel plate shear walls, existing research can generally be applied. However, the same does not hold for the rehabilitation of concrete structures with the system.

In particular, little experimental evidence is available with respect to the anticipated seismic performance of the original concrete frame. As a result, the concrete frame would generally be assumed not to participate significantly in resisting seismic loads, and the rehabilitation strategy would likely be to limit drifts to levels where frame behavior would remain essentially elastic. This is particularly true for older frames that are unlikely to possess ductile detailing and are most likely to be the ones in need of rehabilitation in the first place.

Although different methods of rehabilitating deficient reinforced concrete frames with steel plate shear walls have been proposed, any method must provide a means of transferring forces at the interface of the steel panel and concrete frame members and must consider compatibility issues between the nominal ductility of the existing frame and the ductile nature of the steel infill panel. A research project currently being conducted at the University of Alberta on the rehabilitation of older concrete frames using steel plate shear walls is treating the performance of the concrete frame as a primary consideration. As such, a connection scheme has been devised not only to transfer the interfacial forces but also to provide confinement and shear reinforcement to the concrete columns, thereby enhancing their ductility should plastic hinges form.

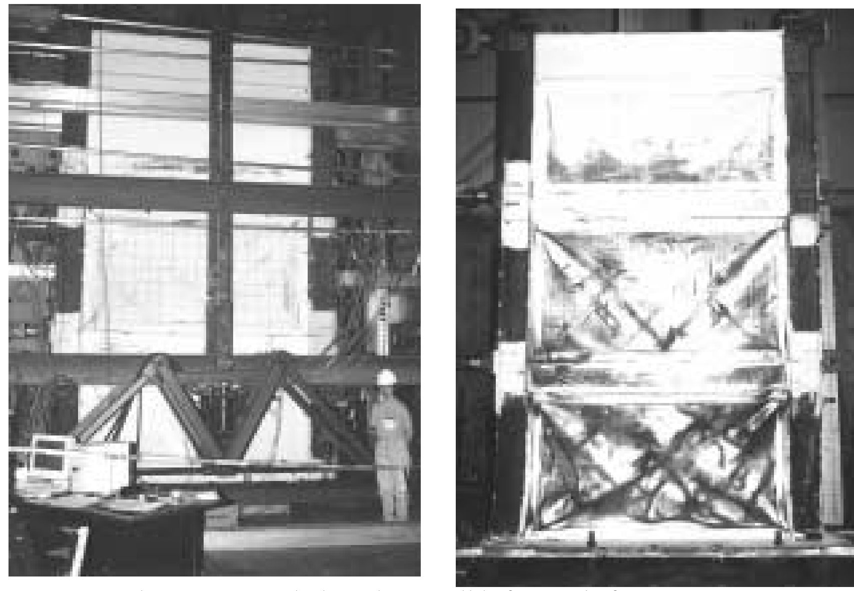


*Figure 4. Three-story steel plate shear wall before and after testing: (left) Before (right) After.*

The steel infill plate is connected to the concrete columns using a series of HSS tube collars, as shown in Figure 6. These connection collars are also, in effect, reinforcing ties with several advantages over internal rebar ties. First, they enclose the concrete cover, greatly increasing the effective area of concrete at the ultimate strength over columns within which only a central concrete core is confined. Second, they have a significantly larger width and flexural stiffness than typical rebar ties, thus improving the efficiency of the confinement mechanism. Third, they provide substantially increased cross-sectional area over typical rebar ties to improve the shear resistance of the column.

Preliminary analytical studies, which are currently being complemented with experimental work, have indicated that significant improvements in both strength and ductility can be expected. As depicted in Figure 6, the spacing of the collars varies according to the expected demand at a particular location in the column. Columns that do not interface with steel panels could also be rehabilitated in this manner to improve their ductility. Future phases of this research program include analytical and experimental studies on the effectiveness of the HSS collars in improving the column shear capacity and ductility and on their effect on overall frame performance.

The use of connection collars, as opposed to the more common drilled expansion or adhesive anchors, has several advantages in addition to the anticipated enhancement of the concrete frame behavior. In contrast to extensive chipping to expose rebar and subsequent drilling and dry-pack grouting, the system of collars is relatively non-invasive and reduces the noise and dust levels, thereby lessening the inconvenience to occupants and reducing installation time. The collar system also reduces the number of trades required for panel installation, since they are simply bolted to the existing columns.

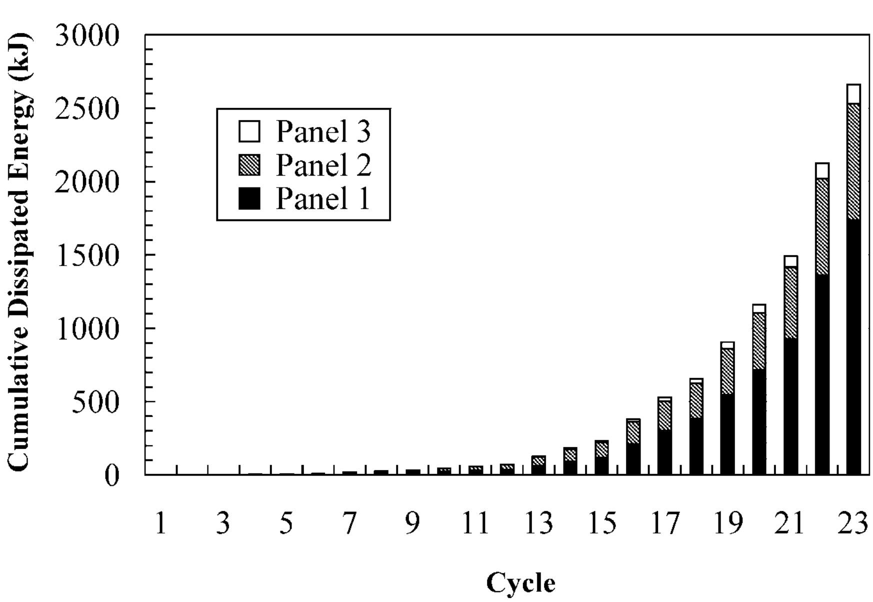



*Figure 5. Energy dissipation in three story steel plate shear wall.*

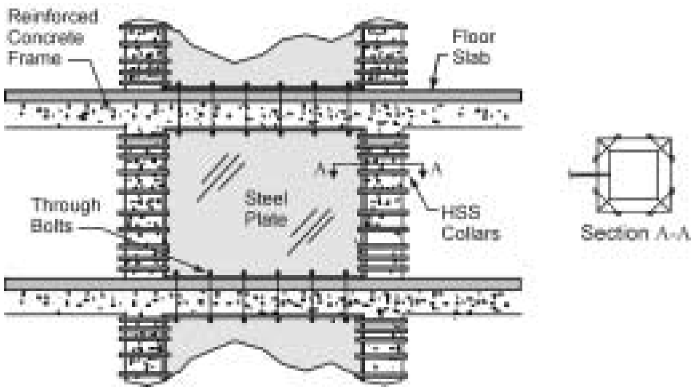



*Figure 6. SPSW rehabilitation of a concrete frame using connection collars.*

## The understudies–stars of future

As with all innovative applications of technology, the technical studies on steel plate shear walls are ongoing, not only to continue the study of the basic configuration but also to develop new ways of optimizing and improving the system. This evolution is resulting in several innovative ways of applying the basic concepts.

Although the columns can be designed to fulfill their dual function of resisting both lateral loads (tension field anchorage and frame action) as well as vertical floor loads, separating these functions may be advantageous. Figure 7 shows a steel plate shear wall with supplementary columns (actually vertical beams) expressly for anchoring the tension field that develops in the infill plate and perhaps also forming a rigid frame with the floor beams. The primary columns could carry only gravity loads, or they could be included as part of the moment-resisting frame. In the latter case, the beam ends could also be detailed as an additional energy dissipating link. As a means of reducing the tension field forces on the anchorage members, a low yield steel that is both ductile and weldable could be considered for the infill plate material.

Mixed lateral load-resisting systems and hybrid steel plate shear walls are also likely to attract more attention in the future. For example, frame–wall systems (where one bay of infill panels is installed into a multi-bay moment-resisting frame) appear particularly suitable for rehabilitation applications, although they could be used in new construction as well. Also, systems of steel plate shear walls with either shotcrete or cast concrete applied against the surface to form a composite panel may provide benefits such as increased stiffness and improved fire resistance.

Another area of future focus that is important for maximizing the economy of the steel plate shear wall system is prefabrication. Fabricating large assemblies in the shop and minimizing the connection requirements in the field appears to be advantageous. As more fabricators gain experience with steel plate shear walls, the most economical techniques will continue to emerge.

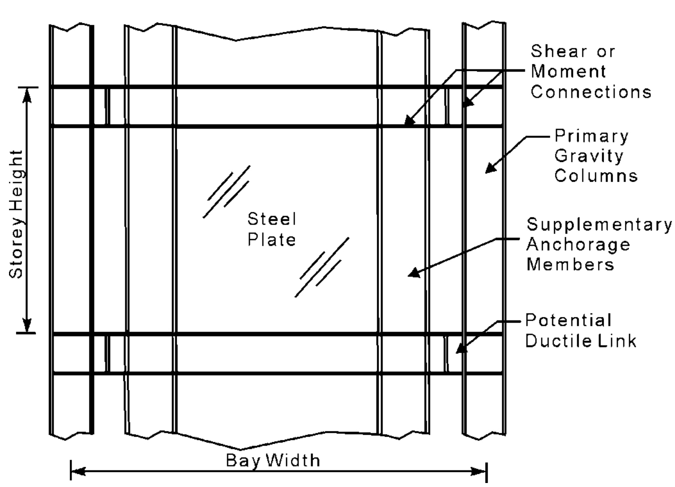



*Figure 7. SPSW alternative with supplementary anchorage members.*

## Curtain calls

The future of steel plate shear walls is bright. A strong history of research and development combined with the rich possibilities of future applications has made the system a sound alternative for building designers. The input of fabricators is now as important as ever to ensure that the system can be built and erected economically. With the new provisions of the Canadian steel design standard nearing the stage of issue and the attraction of new researchers to this area of study, it is anticipated that the use of steel plate shear walls in buildings will increase steadily in the future.

## Additional Info:


- CSA. (2001). CAN/CSA–S16, "Limit states design of steel structures." Canadian Standards Association, Rexdale, ON.
- Driver, R.G., Kulak, G.L., Kennedy, D.J.L., and, Elwi, A.E. (1998). "Cyclic test of a four-story steel plate shear wall," ASCE J. Struct. Engrg., vol. 124, no. 2, pp. 111-120.
- Driver, R.G., Grondin, G.Y., Behbahanifard, M.R., and Hussain, M.A. (2001). "Recent developments and future directions in steel plate shear wall research." Proc., North American Steel Const. Conf., Ft. Lauderdale, FL.
- Robinson, K. and Ames, D. (2000). "Steel plate shear walls: Library seismic upgrade." Modern Steel Construction, AISC, Chicago, IL, (Jan.), pp. 56-60.
- Timler, P.A., Ventura, C.E. (1999). "Economical design of steel plate shear walls from a consulting engineer's perspective," Proc., North American Steel Const. Conf., Toronto, Canada, pp. 36–1-36–18.


*Robert G. Driver and Gilbert Y. Grondin are members of the Department of Civil and Environmental Engineering, University of Alberta.*