The advent of Geomembrane Faced Earth and Rock Dams (GFRD)

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ABSTRACT: Upstream facings to waterproof earth and rock dams offer several advantages. Among the many materials used for decades, modern polymers (Geomembranes) proved to possess substantial advantages and were successfully applied to dams of increasing height. Deformations are inevitable and relevant in high rockfills. Facings should therefore have: low permeability, large deformability and be joint-free. Geomembranes, associated with other synthetic materials, fulfill all such requirements. PVC and HDPE are the most commonly used geomembranes and their properties and applications are described in detail. Design details, like supports, welds, peripheries, protections and anchors are reviewed and described. Significant GFRD dams are listed for reference.

Keywords: dams, geomembranes, upstream facings, polymers, PVC

1 PROS AND CONS OF U/S FACINGS IN EMBANKMENT DAMS

The trapezoidal earth & rock mass of a dam is in far better conditions to resist the hydrostatic load of the reservoir when the waterproofing element coincides with its upstream face.

Figure 1: Nam Ou 6 dam, 88 m high, with upstream PVC facing commissioned in 2015

The entire mass of the dam resists the water thrust and the unit friction resistance required from the foundation is lower than in any core solution. Only the horizontal component of the hydrostatic thrust must be resisted by friction along the entire base and sides of the dam, the vertical component, applies a compression on the fill and foundation and therefore increases confinement, shear resistance and moduli of both fill and foundation.

The grout curtain of an upstream facing dam, forcibly located at the face-to-abutments contact, lowers the piezometric surface throughout the abutment and under the whole embankment. At equal foundation/embankment permeability and curtain efficiency, the pore pressure field is more favorable than in any other scheme.
A uniform rockfill prism has greater tolerance to seepage. Foundation grouting can thus be somewhat less-than-perfect.
The upstream facing scheme allows accessibility to the curtain for inspections, repairs and strengthening.
When acted upon by a train of seismic waves, the body of the dam responds as a single mass of fairly uniform dynamic characteristics. A core represents a discontinuity in response and the upstream and downstream shells may move in counter-phase, at certain moments of the shaking.
From a construction view point, the situation of a uniform embankment lends itself to maximum rate of progress. The grouting of the foundation can proceed as a separate and parallel operation while most of the dam's embankment is being placed. This offers the possibility of saving in completion time. On the other hand, a reinforced concrete plinth is always necessary, while most central core scheme do not require it.
The waterproofing of the upstream face can be installed in stages, this allowing early impoundments or a safer diversion of large floods. Facing installation time must be added, though, to the time necessary to complete the embankment before commission the dam.
From the investment view point, placement of a single material for the whole embankment is faster and entails lower costs. The plinth and the associated longer grout curtain add to the cost.
Considering the service life of the dam, the waterproofing element of an upstream facing is more vulnerable to damages than a central core. Upstream facings though, can be inspected and repaired, in some cases, even underwater.
The upstream facing is better adapted to future increases in height of the dam.

2 CURRENT SOLUTIONS FOR U/S FACINGS EMBANKMENT DAMS

Waterproofing embankment dams with an upstream facing is a scheme adopted long since, appealing for its simple design and fulfilling performance.
Upstream decks evolved also as a result of continued efforts from dams engineers to find materials of higher and more uniform quality to be used as a waterproofing barrier, a vital component of their design.

2.1 *Wood or Shotcrete*

Wood facings on dumped rockfill mark the start of this solution. They were adopted initially in mining districts. Cogswell dam, 85 m high impounded in 1938 is possibly the best example [1]. The embankment was rock, dumped in high lifts. The deck was a 3 plays of fir pine in planks 50 mm thick and it performed with a few flaws for over 10 years.

2.2 *Metal*

Iron facings were used in a few instances. Oxidation and corrosion discouraged their use although Skagway Dam (Colorado) has been in service more than 60 years.
The last such dam, Aguada Blanca (Peru) 45 m high, has been commissioned in 1970 [2].

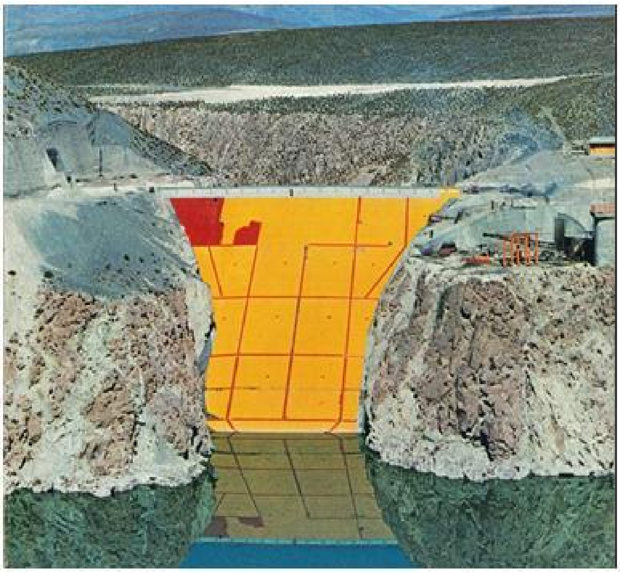

Figure 2: Aguada Blanca ingot iron faced rockfill at 3640 m altitude in the Andes

2.3 *Bituminous Concrete*

Bituminous facings were largely adopted in the past decades and are less frequent today. Since the early examples like Mulungushi (South Africa) facing cross sections and bituminous mixes underwent a large evolution: double deck including a drain layer, and bituminous concrete mixes using standard asphalt cement were simplified to single deck schemes with modified bitumen. In parallel, deck laying pavers, rolling equipments and joints treatment methods were greatly improved. An evolved example was the single deck Montgomery dam (Colorado) 35 m high, commissioned in 1956 [3].

2.4 *Reinforced Concrete*

Concrete facings date back to the beginning of last century. A remarkable example was San Gabriel 2 (California) 115 m high started in 1932 [4]. Later this solution underwent substantial improvements and became widely applied, boosted by an apparently simple design and by an increased confidence of engineers in reinforced concrete. In recent decades, at a time when concrete gravity and arch dams application decreased this got to be the preferred solution. Many of the recent and highest dams like Karahnjukar (Iceland) 190 m and Campos Novos (Brazil) 202 m are of this type. The solution is conventionally referred to as 'Concrete Faced Rockfill dam' (CFRD).

2.5 *Synthetic Materials*

Synthetic Materials (geomembranes) became available to civil engineering since the late 1950s. Their early applications to earth & rock dams date from 1959 [5]. A substantial evolution of polymers, the variety offered by the chemical industry and the introduction of 'composites' (a combination of different synthetic materials like geomembranes with geotextiles), promoted their application especially as refurbishment measure on existing dams. Synthetic facing offer remarkably low permeability and high deformability and were used as the sole waterproofing element in new dams of increasing height: Pitshanulok (Thailand) 20 m high 1978, Jibyia (Nigeria) 26 m high 1991, Bovilla (Albania) 58 m high 1992, Nam Ou 6 (Laos) 88 m high 2014 [6]. Several applications dealt with waterproofing tailings dams.
This design, with service life records exceeding 40 years, is conventionally referred to as 'Geomembrane Faced Rockfill dam' (GFRD).

Figure 3: Murdari GFRD dam waterproofed with wide, pre-welded, bands of polymer ‘composite’: PVC geomembrane + PP geotextile.

3 STRESSES AND DEFORMATIONS IN U/S FACED EMBANKMENT DAMS

During the early successful years after the introduction of CFRD, the pioneers of this solution were adamant in declaring that no computation was necessary. Facing thicknesses, reinforcement quantities and peripheral plinth sizes were recommended only as a fraction of the height of the future dam. Slopes were always step (1.3H/1V). Design practice was influenced by this philosophy and essentially the same cross section was applied to increasingly high dams. Severe accidents however affected in recent years some of the highest CFRD [7], forcing to reconsider the relevance of stresses and deformations in high embankments. Key facts, other than height, are: valley width, abutment steepness, abutment profile and, last but not least, the engineering parameters of the rockfill, like strength and moduli, as well as construction specifications

Figure 4: Damage to a concrete facing due to rockfill deformations (left) and large water loss crossing the embankment, consequence of invisible damages to the facing (right).

The drive toward developing flexible and deformable facing was largely promoted by repeated observations of deformations, joint ruptures and cracking of the slabs of CFRD and by high and increasing rates of water loss in high rockfills. The deformability of synthetic polymer sheets (geomembranes and ‘composites’), plus the possibility of creating a continuous facing by welding, appeared a promising answer to the limited capability of a reinforced concrete facing to survive unharmed the unavoidable deformations of high embankments.

It appears necessary to recall the key facts governing the deformations of a rockfill embankment during construction and under the hydrostatic load of the full reservoir.

Grain breakage is governed by Griffith/Marsal law [8] according to which a rock, characterized by parameter η, shows crushing strength P_a proportional to the log of the size d, the significative diameter of its grains

$$\mathbf{P_a = \eta * d^{1.5} \ (kPa)}$$

Quoting Marsal [9] *“one of the most conspicuous phenomena observed in rockfill masses when stressed, is the fragmentation of the component grains. Partial breakage changes the grain size distribution and appreciably affects the deformation characteristics of the material.”* Marsal defines grain breakage Bg as the sum of positive weight changes $\Delta W > 0$ of each sieve fraction

$$\mathbf{Bg = \Sigma \ \Delta W > 0 \ (\%)}$$

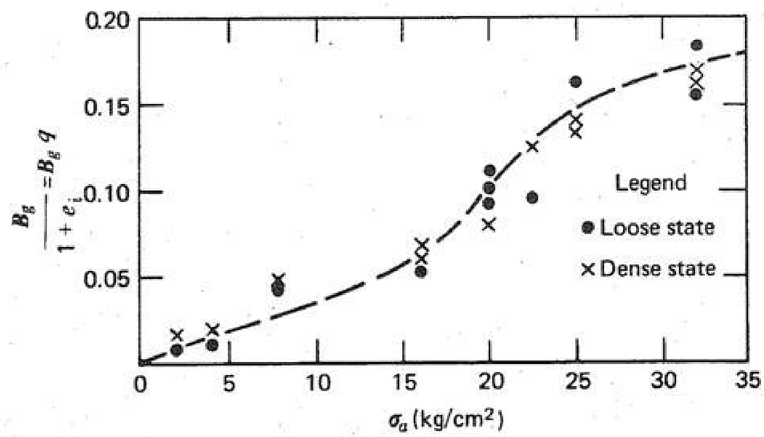

Figure 5: Increase of grain breakage in oedometric conditions as function of the axial stress (from Marsal[9])

For a large spectrum of rocks in oedometric conditions, grain breakage increases with the increase of the main principal stress σ_a. As shown in Figure 5

The stress distribution in a trapezoidal embankment placed in increments is quite straightforward and depends on the height of the cross section, on the embankment’s outer slopes.

During embankment construction, grain-to-grain contact stresses, progressively adjust to the increasing load applied by the superimposed fill. Grain breakage simply reflects the level of principal stresses existing at each contact point. The resulting volume changes are compensated as new lifts are added.

The application of the hydrostatic load on the upstream face of the dam, until then stressed by its own weight only, produce a sizeable rotation of the principal stresses in a portion of the embankment and a drastic increase of their levels as shown in Figure 6.

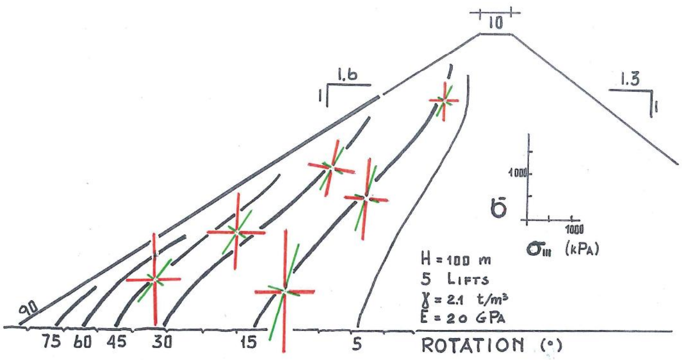

Figure 6: Lines of equal rotation of principal stress directions and principal stress magnitude ‘end of construction’ (green) vs. ‘fully impounded conditions.’ (red)

The rock grains now stressed in different directions undergo further and relatively rapid grain breakage. Smaller fragments can occupy part of the voids existing within the fill. The result is a further, appreciable volume change. The overall volume thus decreases, the unit weight increases and the modulus of deformation may change. The gross permeability of the fill tends to decrease.

Grain breakage and volume changes are clearly larger under the upstream lower face, where rotations and stress increase are greater, and lesser near the crest. They fade out approaching the dam’s axis. In addition they change in amount as one moves from the centerline toward the abutments.

The field of ensuing deformations is highly differential and multi-directional. Differential deformations become of more impact the higher the dam, the narrower the valley and the steeper the abutments. Deformations are inversely proportional to the fill quality (shape and diameter of the governing size of the rockfill, strength of individual grains), to the level of compaction and to the changes between initial and final grading.

As a consequence the facing suffers both contractions in all directions and extensions, on both sides near the crest, related to the bowl-shaped displacements in a direction normal to the face.

Quoting Terzaghi [10] “… *settlements of the facing under water load, in a direction at right angles to the upstream slope, increases … in proportion to the square of the height of the dam … and the thickness of the layer subject to compression* [slabs] *increases in simple proportion to the height … difficulties of adapting the concrete facing to the settlements … increases rapidly with increasing height…*”

Another source of post-construction deformations is the saturation of the lower part of the embankment in which conditions rock-to-rock contacts became fragile and collapse.

Quoting Mould [11] “*in general water and water vapor is the primary agent in promoting fatigue* [crushing failure] … *and the interaction appears to involve primarily the neutral water molecule and the SiO_2 network* [of the rock] …[12] *aging effects … can be accounted for an increase of the radii of the tips of the abrasion cracks in the* [rock grains] *surface*”].

Quoting Wilson [13] …’*even the hardest and more competent rockfills are likely to be more compressible than clay cores and the first saturation increases the compressibility of all rockfills*’.

The initial grading and placement methods of the rockfill plays a substantial role. Clearly coarse, dumped in high lifts and not sluiced embankments can potentially suffer the largest deformations. Well graded, spread in metric lifts, compacted [15] and sluiced embankments should be expected to undergo the least deformations. In all cases the height of the embankment is a major cause of large deformations.

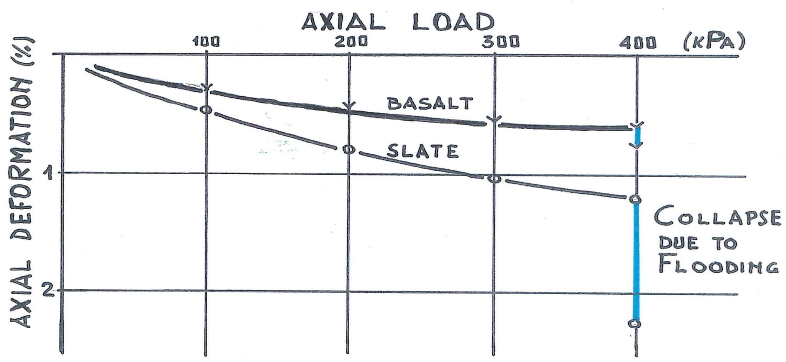

Figure 7: Effects of saturation on different types of rock in oedometric conditions (from Sembenelli, Biondani [14])

There is a large data base on deformations of upstream face of rockfill dams for different grading, materials and construction methods [16] [17]. Understandably, not all sets of measurements are equally reliable. Monitoring data available in literature suggest the following ranges for the deformations to be expected in the short term for a high rockfill dam with impervious facing (H= maximum height, L=crest length):

– bulge deformation (outward) of the lower face under dead weight 0.004 H to 0.009 H
  Bulging may affect the lower 20% of the facing.
– bowl deformation (inward) of the entire face under hydrostatic load 0.001 H to 0.025 H
  The bowl is deepest near the center of the facing at 40 % of the dam height .
– converging displacement parallel to the axis toward the face centerline 0.001 L to 0.003 L
  Extension zones cover 20 % of the crest length from both abutments.
– crest movement (downstream) upon impounding 0.003 H to 0.017 H
  Crest movements during the early stages of impoundment are toward upstream.
– crest settlement upon impounding 0.005 H to 0.020 H

Long term deformations may be up to 1/3 of those produced by the first impounding and can continue for decades.
Extrapolation to a new dam are never straightforward, especially if the engineering parameters of the rockfill under load, and in presence of water, are not accurately known from large scale tests.
To produce facings of conventional materials, compatible with such deformations (extensions, displacements, and compressions) joints of different, often elaborate, design have been devised and adopted. Clay cover, in form of an embankment blinding the lower part of the facing and peripheral joint, have become a standard part of design (adding to the cost) but nothing of the like proved effective when rockfill deformations drag the structural elements of the facing and dislodge the joints.
The list of facing distresses is long and different for the different types of facing. A umber can be described as performance decay due to aging, erosion, deformations. Far more can be described as cracking or compression failures (see Figure 8). This second type of distresses has grown impressively in recent years with the increase in height of CFRD, designed with insufficient attention to the rockfill characteristics and to the deformation processes related to placement, impounding and saturation.

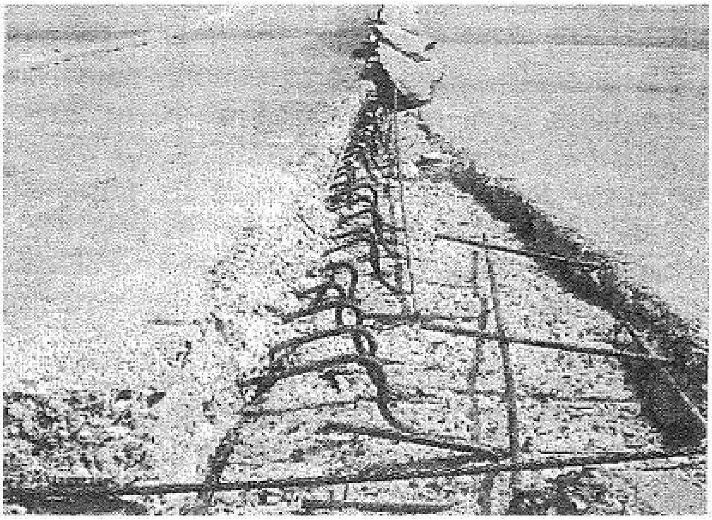

Figure 8: Concrete facing failed in compression along a vertical joint (from Pinto [16]).

Progress of numerical analyses and of 3D modeling has lured many designers away from experimental measurements of the rockfill, modulus, shear resistance and grain breakage under increasing loads. The result of an advanced analysis purely based on literature or on extrapolated data can be surprisingly poor.
The now customary 1.3H / 1V slope of CFRD facings appear too steep and enhancing all deformations, regardless of the type of facing. Flatter facing slopes are usually preferred in GFRD. Slopes 1.5 -1.6H / 1V have been used in several dams with positive results and are the recommended design for high quality rockfills. Weaker rocks may require flatter slopes.

4 REQUIREMENTS OF MATERIALS FOR U/S FACINGS

In the light of what has been discussed above, the key requirement for a performing facing material, to be installed on an upstream faced dam, are: -i) permeability to the lowest level, the primary requisite of any lining -ii) weldability to avoid structural joints -iii) deformability in extension and compression to eliminate the need for elaborate joints and all measures devised as a backstop in case of partial joint failure -iv) puncture resistance -v) endurance to UV rays. Conventional materials used by civil engineering miss one or more of the above. Synthetic materials offer the best combination of them all.
For upstream decks the aim of the profession was also that of installing materials of higher and uniform quality. Even in presence of strict QC/QA specifications, civil construction industry cannot assure that a given material is supplied and placed with uniform characteristics. In this respect the quality and uniformity of products from the chemical industry is by all means better than any alternate, traditional product particularly if made on site. The way in which polymers are produced is a unique advantage of a synthetic facing (geomembrane and ‘composite’).
Geomembranes can be welded (at the factory, in a shop or on the dam’s face) and welds can be tested. The absence of joints over the entire geomembrane facing is another plus with respect to jointed facings for which a generalized testing is hardly feasible.

5 ADVANTAGES SPECIFIC OF GEOMEMBRANES AND COMPOSITES

In relation to a deformable support geomembrane facings offer never-seen-before performances in force of their proven capability of deforming in extension. The industry is offering steadily new and better polymers. Failure strain differs from polymer to polymer: for PVC this is in excess of 150 /200%. PVC coupled to a geotextile (?composite') usually has a lower failure strain but always in the range of 50%. HDPE has a failure strain in extension close to 10%. In the large scale this means that a geomembrane or a 'composite' facing can follow the elongations or contractions of the rockfill face under the reservoir impounding and drawdown cycles. In the small scale, the large strain potential of polymers can drape important irregularities of the support (see Figure 9). Water tightness remains unchanged.

Figure 9: (left) A large (0.8 m diameter), high pressure (up to 300 m of water column), vessel to test geomembranes, 'composites' and systems of synthetic materials on different supports, on a 1:1 scale. (centre) A typical aggressive support (50 mm crushed). (right) A 1.5 mm thick HYPALON geomembrane over a 35 mm crushed basalt under a 200 m water column (the grid squares are 50 mm).

Geomembrane facings permit fast placement rates in the order of 1000 to 1500 m2/day for 2 m wide rolls and up to 2000 m2/day for 6 m wide rolls. The work required along the plinth often commands that overall placement rate.
A geomembrane deck can easily be placed in stages, following the embankment progress (and the progress of the peripheral plinth and of grout curtain). This is particularly important for tailings dams which are raised in several successive steps (see Figure 10). In case of cofferdam overtopping, a partial deck can work as a back-up cofferdam, increasing both the capacity of the diversion and the safety of the jobsite. Finally, a partial deck, installed at an early stage of construction, and a partial impounding of the reservoir, may help containing the deformations of the embankment.

Figure 10: A staged application of a geomembrane facing on a tailing dam in hot climate (Iran)

Ice does not adhere on most polymers and an exposed geomembrane facing is usually slightly affected by ice sheets when the reservoir is in cold climate (see Figure 11). Yet, thick and dirty ice may require special anti-wear measures.

Figure 11: A PVC ‘composite’ facing under severe winter conditions (Norway)

An exposed geomembrane facing can be inspected readily when the reservoir is drawn down, also for the portion permanently underwater. In most cases, particularly if the selected polymer is PVC, repairs can be done underwater with patches fixed with steel profiles and bolts, or with special adhesives.

6 POLYMERS FOR GFRD

Chemical industries have developed a number of resins and polymers since the first half of last century.

While the early synthetic materials (1920s) were rigid (Celluloid, Galalite, Bakelite). Deformable thermoplastics came to the market in the 1940s as Polyvinylic (PVC), Polyamidic (Nylon), followed in the 1950s by Polyolefinic (Polyethylene) (Polypropylene) and Synthetic Rubbers (Hypalon) (EPDM). Each of the above products offers advantages and has drawbacks.

Civil engineering has taken interest to some of them and used them in different ways. The interaction market-designers resulted in the adoption of different polymers in the course of time. Pressure vessel tests, design analyses and the results of applications, highlighted the paramount characteristics of geomembranes, for applications in dams: high deformation potential (failure strain in tension), easy welding, dimensional stability against temperature changes, durability in UV and other aggressive environments. A rationale choice of the polymer best adapted to the specific needs of the project should be done taking into account a combination of the above.

After a number of applications, by far today's most adopted polymers are PVC and HDPE. The potential of a polymer to respond to the project needs also depends from the percentage of the resin used in the production of each of them. While HDPE is produced with 95% of resin (whose formula is by definition non modifiable) and 5% of lamp black, PVC contains only 50% of resin added to an equal amount of plasticizers, stabilizers, pigments, mineral charge and lubricants. This to say that different Manufacturers can produce different PVCs in which one or a few specific properties can be enhanced to better suit a given project needs. Some typical characteristics of PVC and HDPE are given below:

Polymer	Type	Unit weight	Coefficient of thermal expansion	Failure strain	Melting temperature
		kN/m3	-	%	°C
PVC	plasticized	13	7 $*10^{-6}$/°C	< 200	160
HDPE		9.5	85 $*10^{-6}$/°C	ca. 10	135

Figure 12 below shows the deformability characteristics of different PVC and of HDPE. Actually, state-of-art design of a synthetic facings use multiple layers of different products (geomembrane, geotextile, geodrains, etc.).

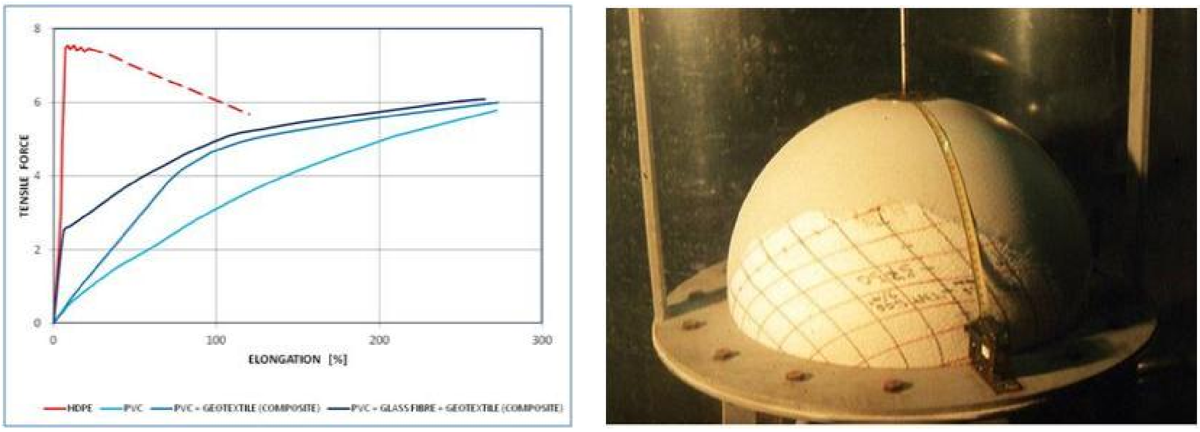

Figure 12: Stress–strain curves of different PVC and of HDPE (left). Extension potential of a PVC 'composite' The geomembrane has not failed at ca.200% in 3D extension (right). The geotextile failed at about 50% strain (curtesy CARPITECH)

7 WELDING OF GEOMEMBRANES FOR GFRD

As said above, welds are possibly the most important part of a facing composed of strips produced in metric widths. Obtaining performing welds, with minimum probabilities of flaws, is of vital importance for any GFRD dam. Practice has shown that high quality welds through an easy and reliable process is key to success. Difficult or unreliable welds, or welds that may pop open after a short while, disqualify the best design and an optimum project.

Dimensional stability of the polymer or composite selected for the deck is of absolute relevance. In a roll of liner which changes appreciably its dimensions with temperature, exposure to sun may induce excessive elongations turning into heaps, ondulations and localized stresses. Cycling temperatures may result in fatigue failures. Built-in stresses will be inevitably locked when a roll placed a few hours before is welded to a next roll of different temperature. The welding itself (which requires temperatures of ca. 350 °C, depending from the outside temperature) generates localized thermal dilations that not always are uniform and, when the material cools down, turn to differential stresses that are permanently locked-in. Figure 13 shows the effects of an excessive thermal expansion.

Welds can be done by fusion (induced by hot wedges or hot air), by extrusion of a small amount of a newly added material and with glues.

Figure 13: Elongations and distresses of an HDPE liner produced by temperature changes

The 'state of the art' welding of PVC and HDPE, the most commonly used materials in GFRD, is obtained with double track hot wedges or hot air blowers bringing the temperature along the welds to fusion point. Prior to heating, dust and foreign matters shall be thoroughly eliminated from the surface. Immediately after heating the 2 sheets to be welded must be pressed with 300 kPa for a 2 mm PVC sheet, 400 kPa for a 4 mm PVC sheet and 2000 kPa for a 2.5 mm HDPE sheet. When the underneath of the weld is accessible, automatic welders can be used (like LISTER Twinny). The operator sets the desired pressure on the weld and the machine automatically controls its speed. When the underneath of the weld is not accessible, welding is manual with a simpler machine (LISTER Drive) and the pressure must be applied by the operator (see Figure 14).
PVC is easily welded. The substantial rigidity of an HDPE makes often difficult to apply the necessary pressure over the seam, especially in the field, temperature induced elongations of the HDPE may result in fish mouths hindering the welding to the adjacent roll.

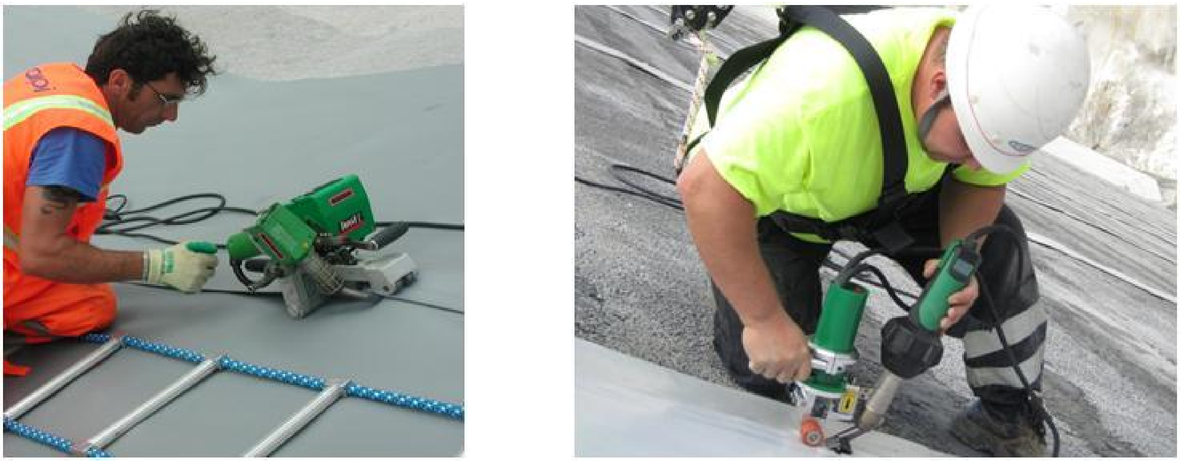

Figure 14: (left) Modern automatic, double track and (right) single track, hand-held welders (courtesy CARPITECH)

Detail welds must be done with air blowers and rollers for PVC and with the addition of an extruded bead of new polymer for HDPE.

All seams must be part of a quality control program. Part of a high quality weld requirement stay in the possibility of performing reliable checks. Back in the mid 1970s, in connection with HYPALON which requires to be vulcanized and is indisputably difficult to weld, the author devised the 'double track' weld: 2 parallel, linear, separated welds are done instead of a single one. A small rectangular void remains between the two welds and the two superimposed sheets of polymer. To make sure that the pipe remained open and continuous, a cotton wick was placed between the 2 welds in the early tests. Soon this was seen not to be necessary and was eliminated making the double track weld a lot simpler. The entire length of a double track weld can be tested for impermeability by closing the non welded gap at both ends and pressurizing the void with air (see Figure 15). Once the weld cooled down, the gap between the welds is blown into a small hose at 2 bar and the air pressure should not decrease more than 10% (creep of the geomembrane) in 5 minutes time. Being air more difficult to contain than water, the hydraulic performance of the seam is hence checked. Manual and detail welds are checked with soap solution and vacuum chamber. Samples of the seams are systematically cut-out and tested in the lab with tension and pealing tests.

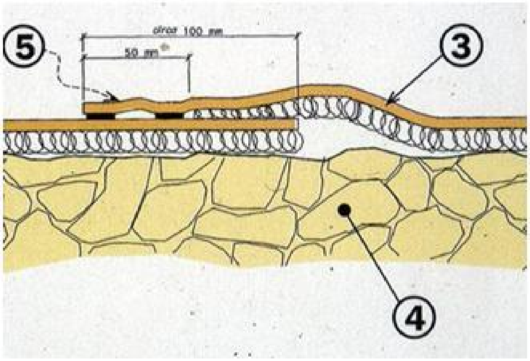

Figure 15: The concept of double track weld (left) and the air pressure test underway on double track seams (right)

8 PERIPHERIES FOR GFRD

The peripheral connection to the foundation or to adjacent structures, is essential for a performing facing. The periphery of a GFRD is conceptually the same as engineered long since for concrete and bituminous facings. It consists of a flat reinforced concrete slab (plinth) poured directly on the sound rock of the riverbed and abutments. Placing the plinth in contact with the abutment along an inclined line require some excavation and rock preparation. Along some portions of the perimeter, the plinth may be substituted by a concrete cut-off.

The plinth also represents the cap of the deep grout curtain and a convenient working surface for drilling and grouting works. It is recommended (although not always done) to anchor the plinth to the rock with grouted re-bars and to treat the rock under the plinth with shallow consolidation grouting before installing the deep grouted curtain. Possible clogging and contamination of the adjacent fill of the dam, particularly the free-draining, fine rock bedding under the facing, impose that all grouting operations be carried out before casting the curbs and placing the fine rock behind. When the possibility exists for the plinth to crack during its service life, the plinth is better covered with the geomembrane of the facing.

Practice has shown that a few geometrical requirements are desirable for the plinth. In the first place the contours of the plinth should be the prolongation of the contours of the facing with no kinks.

Second a short (200 mm at least) sub-vertical face, should be provided to permit a mechanical, watertight connection between facing and plinth. Connection should better be located along the inner edge of the plinth.
Different types of connections have been developed along with the several applications engineered and tested in the past 40 years.
The connection of the facing to a concrete structure or to the plinth is made of two parts: the connection proper and a deformable arrangement: the 'fold'.

8.1 *Mechanical Waterproof*

To fix the watertight component of the composite onto the concrete of the plinth. This requires a previous smoothening of the concrete surface with abrasive mole. Hence stainless bolts are fixed to the concrete with two-component resin. The composite is then set in place after creating a selvedge without geotextile. Ripples are eliminated. Hence a smoothening resin is applied and the geomembrane is holed through at the exact location of the bolts. A rubber gasket is set over the geomembrane covered by a stainless steel profile. The nuts are closed. A second tightening of the nuts is done 24 hours after and finally each nut is checked with a dynamometric wrench (see Figure 16)

Figure 16: The mechanical watertight fixation of the composite to the concrete plinth

8.2 *Mechanical*

It is obtained as the one above generally without the smoothening operations and without rubber gasket.

8.3 *Insert*

The concept of the ‘insert’ periphery is simple: a slot is left into the concrete along the periphery (plinth). The slot must be wider whenever a structural joint must be crossed.
The insert arrangement has been first developed by the author to cross expansion joints of concrete structures onto which the facing had to be fixed (see Figure 17).
Later the ‘insert’ proved an interesting alternative to mechanical connections also for general applications.

Figure 17: Early tests to cross structural joints with a geomembrane from which the 'insert' solution was born

A band of geomembrane is glued and sealed into the slot. Both a glue and a sealing mastic must be used to set the geomembrane in the slot and their final profiles must be such that the hydrostatic pressure will press the geomembrane against the concrete and close the joint. The slot can be 80 to 100 mm deep and 30 mm wide, tapered. The width of the geomembrane band is larger than the slot depth as much as required by the specific geometry of the connection (see Figure 18).

Figure 18: Close–up of a training 'insert' fixation (left) and an 'insert' on the top of a plastic concrete cut-off (Jibiya dam) (right)

The end of the geomembrane band is later bent and welded to the facing. The first such application dates back to 1989 at Cixerri dam (Italy). The 'insert' concept can be applied also along the top of a concrete cut-off.

8.4 *Fold*

According to the geometry of the plinth and the foreseen deformations of the face, the connection facing-to-concrete shall make allowance for deformations. The 'fold' shall allow the face to depress and move away from the plinth, without transferring substantial tension to the periphery or to the composite. This is obtained with an extra length of geomembrane. The design of the fold (see Figure 19), must be studied case by case and it generally use both deformable fill (fine uniform sand) and low friction materials like LDPE and lubricants.

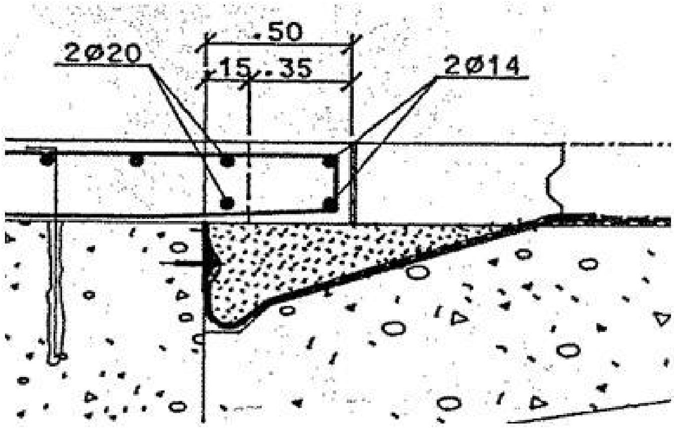

Figure 19: A typical 'fold' along the facing-to-plinth contact (Bovilla dam)

9 PROTECTED VS. EXPOSED GFRD

Polymer facings have been designed both protected by different covers and without protection (exposed). There are good reasons for the adoption of either solution. Both protected and exposed solutions went through a long development period as either one requires finding appropriate solutions to specific problems.

9.1 *Protected Facing*

It solves the problems of instability against gravity, wind and wave actions, related to the limited rigidity of facings based on synthetic materials when installed on steep slopes. A protected polymer is shielded from UV radiation and this decreases its aging rate. Protection limits occasional damages coming from natural events or human insults.
Such considerations have resulted on protective covers, of different weight and rigidity, to be part of most early designs. Protections, in themselves a source of damages, almost invariably make difficult detecting leaks whose location requires the removal of the cover, always an awkward job.

9.2 *Exposed Facing*

It offers, on the other side, several advantages like: the possibility of full and direct inspections, the possibility of repairs (even underwater), the elimination of potential damages derived from the different operations required by the installation of whatever cover could be decided. Savings corresponding to the protection cost and the relevant additional construction time also have a sizeable weight and makes an exposed facing appealing.
Yet, the limited rigidity and insufficient stability of a facing based on synthetic materials requires a performing anchorage to the embankment before an exposed solution can be considered.

10 OVERLAYS FOR PROTECTED GFRD

Protection must be adequate to resist whatever outside conditions are taken into account in the design. They must be designed for long term compatibility with the facing, taking into account the expected short and long term deformations of the embankment as well as potential uplift pressures. Important in design are the measures to be taken to safeguard the facing while the different operations required by the selected protection are carried out.
Protected facings can be of different classes.

10.1 *Light*

It is typically obtained with a nonwoven, separator and de-coupling geotextile. A polymer reinforcement in form of a net is superimposed. Finally a fiber reinforced shotcrete crust is applied. Geotextiles should be nonwoven, continuous filament for mechanical resistance and Polypropylene fiber for compatibility with the alkali of cement. Convenient weep holes or joints must be provided to limit uplift in case of draw down (see Figure 20).

Figure 20: Light geotextile + shotcrete protection on a relatively flat slope

10.2 *Medium*

It is typically obtained with a nonwoven separator and de-coupling geotextile plus a thin fiber reinforced concrete slab, cast-in-place. Geotextile requirements and behavior are as described above. Size and shape of the slabs must be selected with care, considering potential deformations of the support and possible concentrated loads over the slabs. The conglomerate finer fraction impregnates the top of the geotextile which becomes intimately connected to the concrete. For thin castings the geotextile works as an efficient reinforcement. Tongue-and-groove joints are necessary between adjacent slabs and the position of the groove is better reversed at the midpoint of each joint. The tongue-and-groove requires a minimum thickness of the slabs in the range of 60 to 80 mm. A triple ply geotextile inserted along all vertical joints provides both joint play and venting of uplift pressures (see Figure 21).

Figure 21: Medium unreinforced, cast in place concrete slabs protection

10.3 *Heavy*

This is typically obtained with a nonwoven separator and de-coupling geotextile and a fiber re-inforced concrete slab, 200 to 300 mm thick, cast-in-place (see Figure 22). The same criteria described above govern size of and connections between slabs and venting of uplift pressures. Some of the slabs may require reinforcements and dowels may be required along some of the joints. A typical example is Bovilla dam [18].The protected solution makes possible using any width of geomembrane rolls on the face. Single rolls, 2.1 m wide, are often used in protected facings.

Figure 22: Heavy cast-in-place concrete slab protection. Reinforcing steel may be necessary on some slabs and dowels may be required along some joint

Heavy protection has also been achieved with precast, steel reinforced concrete slabs. Along with thickness the size of the slabs can be decreased but placement of large and thick slabs may become difficult in case of high facings and dangerous for the integrity of the facing in case of mishandling. Heavy protection may be the only option on steep slopes.

11 ANCHORS FOR EXPOSED GFRD

An exposed facing may be damaged by wind and wave actions, by stone falls in narrow can-yons and by human and animal insults. Figure 23 shows how destructive wind damage can be when anchoring has not been foreseen.

Figure 23: Total destruction of the geomembrane and of the underlying geotextile by wind. Anchors were not installed

Wind and wave damages can be avoided by anchoring the deck into the dam's embankment. Anchoring a flexible and inherently weak cover into a granular surface is not straightforward and the design of anchors has evolved through many attempts and improvements.
Early attempts were based on polymer tongues embedded in the embankment as this was being placed [19]. Polymer tongues were placed with a precise geometry (aligned or alternating) on the upstream edge of the fill with a certain length hanging on the slope and a couple of meters embedded flat over the lift (see Figure 24). This last portion became clamped into the fill when the following lifts were placed and compacted. The facing was unrolled over the slope and welded over the protruding portion of the tongues. A single tongue was used to fix 2 adjacent rolls of polymer. A butt strip was finally welded to unite adjacent rolls.

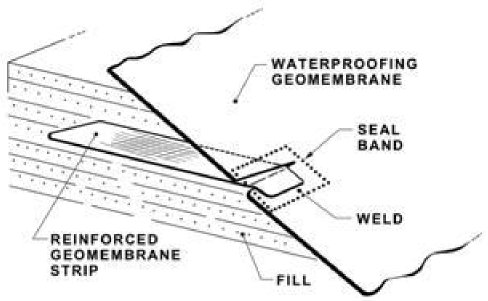

Figure 24: The early concept of anchors in a granular fill (left) and the first such application at Alento cofferdam (Sembenelli [19]) (right)

In some cases the anchoring of the facing to the embankment was done with bolts inserted in the granular fill once the rolls in place and welded. The bolts were either grouted into the fill or simply mechanically engaged ('duck bolts'). Holes had to be cut through the facing to pass the bolts and the water tightness ensured with discs of polymer welded all around to cover the nut. However a large number of penetrations interrupting the continuity of the deck, is not desirable.
The most recent solution, adopted with satisfaction in a number of large and recent projects is based on curbs of porous concrete laid before spreading and compacting each lift. Curbs are similar to those first used at Ita dam in the year 2000. Curbs must be pervious i.e. made with no-sand concrete, typically 80 kg $_{cement}$/m^3 and 1 - 2 MPa unconfined compression strength at 28 days. For geomembrane facings, curb geometry and details have undergone a notable evolution. Past experiences showed that curbs must interlock with each other. The latest design of a curb face calls for compressible joints along a single curb and between superimposed curbs allowing compression of the face in both the vertical and the lateral directions. Joint play and spacing should be such as to allow the deformations obtained with a 3D predictive modeling of the embankment deformations based on real rockfill engineering parameters. Geotextile reinforcements should be foreseen near the curbs to avoid that the embankment's lateral pressure might dislodge one curb and create sharp overhang on the face, which may damage the facing. Curbs have a trapezoidal cross section and the composite tongues are nailed near their base. PVC tongues are long enough to cover the entire curb and to overlap the tongue below (see Figure 25). Each tongue is welded over the previous one thus forming a continuous band of composite anchored to the curbs every 4 - 8 m. Tongues are aligned on vertical planes and spaced a multiple of the width of a single roll (see Figure 26). Their usual spacing is 6 m. say the width of 3 rolls of composite.

The dam's face created by the superimposed curbs must be sufficiently smooth and regular so as to provide a bedding over which the composite might take the highest hydrostatic pressure and move laterally to accommodate for the deformations of the rockfill.

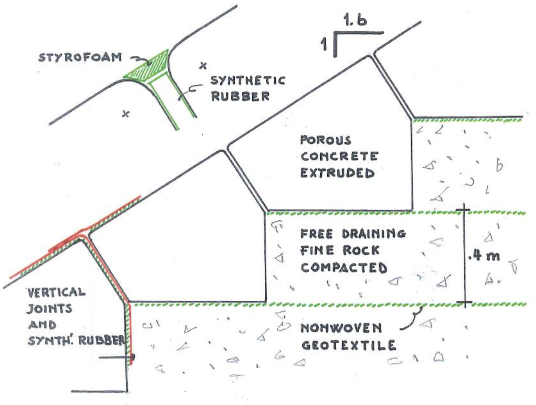

Figure 25: Evolved design of curbs to support a composite deck and hold anchor strips (left) a close up of a curb freshly extruded while the lowest composite strip is being placed (right)

The curbs are normally backed by a zone of fine grained, uniform rock providing a cushion absorbing potential non-uniform spreading/compression of the rockfill and working as a back-drain. Proper drainage requires obviously that the face pervious cushion be connected to a wide and assuredly pervious drain along the dam's foundation.

Recent practice calls for pre-welding of 2 to 4 rolls and the preparation of wider (4 to 8 m wide) bands that are finally placed on the dam's face and welded to the anchor strips. Pre-welding should be done in a protected, dust-free and controlled humidity shed (see Figure 26). Shed weldings can be done with the double track type of seams tested on 100 % of their length with compressed air and therefore certified. Pre-welding 3 rolls reduces the seam length to be done in the field to 50 % of the total. Greater distances between tongues would reduce the field seams even more (pre-welding of 4 rolls would allow to have only 40 % of field seams and 60 % of shed seams) but 8 m wide bands are usually difficult to handle and the anchoring that can be counted on with widely spaced strips may not be sufficient to resist the expected wind and wave forces. The side seams (2 for each band) must be done on the slope and can only be done with a single track weld.

Figure 26: Shed pre-assembly of 3 rolls into a 6 m wide band (left). Seams are automatic and tested on the 100% of their length. Placement on the face of the dam of a 6 m wide band. Field welding is limited to the 2 side seams onto the anchor strips (right)

The smooth concrete face resulting from the casting of curbs may in future expand the application of composites using special adhesives instead of anchor strips. Such application has been done, so far, only on the vertical face of the concrete arch dam Zollezzi (Italy 1993) [20] (see Figure 27).

Figure 27: Pull-out tests on adhesive (left), (centre) measurement of traction resistance of a geotextile glued to concrete and Zollezzi arch dam lined with a glued PVC composite (right)

An interesting provision to counteract wind damages, is suction applied under the facing. Suction is obtained with the wind itself flowing into a set of swinging Venturi tubes mounted on the crest and is distributed under the geomembrane liner by means of fans of perforated pipes. This solution was first proposed by the author in 1983 for Jebel Dzioua reservoir for the water supply of Oran (Algeria). A recent such application has been done at Gilboa upper reservoir (Israel).

12 SIGNIFICATIVE EXAMPLES OF GFRD

Tenths of new rockfill /gravelfill (and earthfill) dams waterproofed with a thin geomembrane or ‘composite’ facing of different synthetic materials (mostly PVC) are in operation worldwide and perform to satisfaction, in some cases since more than 25 years. Significant dams that mark the development of this design concept are listed below:

Name	Country	Height	Facing slope	Polymer	Thickness	Exposed	Protected
		m	H/1V		mm		mm
KURIYAMA 1988	Japan	48.5	<3	PVC	1.5		YES granular
JIBIYA 1989	Nigeria	22	3	PVC reinforced	2.1		YES 80
BOVILLA 1996	Albania	58	1.55	PVC composite	3		YES 200/300
SAR CHESMEH 2008	Iran	40	1.15	PVC composite	3	YES	
MURDARI 2013	Albania	31	2	PVC composite	3.5	YES	
NAM OU 6 2015	Laos	88	1.6	PVC composite	3.5	YES	
RUNCU u.c.	Rumania	98	1.6	PVC composite	3.5	YES	

Today proven experience can be applied to 100 m high dams. All indications are that GFRD concept should be considered for future dams of greater height.

13 GEOMEMBRANES TO REFURBISH EXISTING DAMS

Several existing dams of different design have been waterproofed or refurbished with the application of geomembranes. To date more than 300 dams hold their reservoirs thanks to geomembranes of different polymers used in several different ways and on widely different supports.

Early applications of geomembrane facings as the impervious element of an earth & rock dam date back to 1959. A wider application of this solution started in the 1970s and evolved from low head reservoirs to medium height dams to high dams [5]. The record high is to date La Miel 188 m (Colombia).The majority of such facings are PVC composites.

Different techniques were used to hold the new facing on to the old structure. By far the most used was a mechanical anchor-and-tensioning profile patented by Carpitech (see Figure 28) applied in more than 150 cases. Much less frequent was the use of bolts.

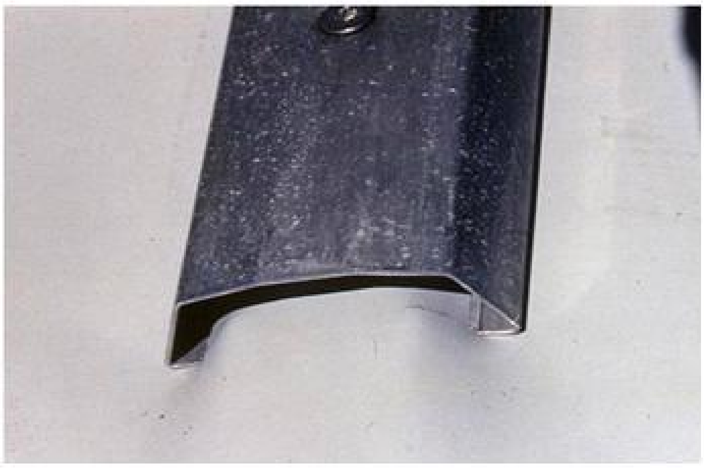

Figure 28: The Carpitech patented anchor-and-tensioning system consisting of 2 stainless profiles pressed onto each other and screwed to the support. A PVC strip will cover the top U profile and will be welded along both edges, to waterproof the anchor.

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