Some times there occurs a need to carry out a joint between the asphaltic concrete core and a concrete lateral wall or some other concrete structure. Legadadi dam drainage pipes positioned above the upper and lower blanket. By means of transverse walls spaced at 60 m, the space between the two asphaltic concrete cores below the gallery is divided into sections in order to obtain easier locating of possible defects.

Figure 14.37 Joint of the asphaltic concrete core with the control gallery, Mora de Rubielos dam (2005, Spain). (AC) Asphalt concrete core; (F1) upstream filter layer; (F2) downstream filter layer; (G) gravel in the dam shells; (AM) asphalt mastic.

Figure 14.38 Joint of the asphaltic concrete core with the foundation, High Island dams (Hong Kong). (1) Asphaltic concrete core; (2) transition zone; (3) control gallery; (4) concrete foundation slab; (5) upper and lower asphaltic concrete blanket; (6) filter material; (7) upper and lower drainage pipe; (8) asphaltic mastic; (9) rockfill; (10) grout curtain.
(Ethiopia) contains a constructed joint of an asphaltic concrete core with the adjacent buttress of a concrete dam, Figure 14.39. Namely, a part of the buttress concrete dam has been constructed as a rockfill dam with asphaltic concrete core, because of unfavourable geological conditions on the ground of one section. The asphaltic concrete core in the zone of the joint is thickened, while the contact with the concrete structural element is inclined and broken, aiming at increasing its length, which reduces the danger of contact seepage. The coating of asphalt mastic (2) is a customary measure at the contacts concrete-asphaltic concrete, as well as the control manhole and gallery (6 and 5 in the sketch). The joint as it is shown in the sketch, should be watertight and without waterstop (3), which has been incorporated as an additional safety measure (Tanev, 1985). The joint between the asphaltic core and roller compacted concrete part of the combined dam Foz do Chapecó (Brazil), completed in 2010, was solved in similar fashion, without a waterstop. The RCC wall at the joint was inclined at a ratio 3(V):1(H) (Saxegaard, 2012).

14.2 OTHER TYPES OF NON-EARTH CORES

14.2.1 Concrete core walls

The application, construction, and characteristics of concrete core walls have already been considered in the chapter on earthfill dams. In the case of rockfill dams, they have been relatively often used in the first three decades of the twentieth century mainly for dams of lower height. In the next 50 years, they have been abandoned because, in a great number of cases, there have come about heavy damage to the watertight element and loss of its functionality due to the impossibility for it to adapt to the significant deformations of the uncompacted rockfill, as well as owing to the transfer of stresses from the embankment into the concrete wall. These would be caused by the exceptionally different deformation characteristics of the two materials. Similar to the case of the concrete facings, the new structure and construction technology of dam’s body of rockfill, placed in thinner layers and compacted by means of vibratory rollers, creates the possibility to reconsider this partly forgotten watertight element,
The advantage of an internal core made of plastic mass, in relation to the appropriate construction in the form of facing, described in Chapters 10 and 13, is that it is not subjected to external influences, to which the delicate construction is sensitive, so it is usually necessary to provide specific protective construction.

The grout core walls have a significant advantage in the fact that they are constructed once dam's body has already been completed, and in case of need of a repair, it is possible to perform additional grouting, without emptying the reservoir. Yet, this construction is justified only in certain specific conditions and cases.

An interesting example is Atbashinskaya dam built in 1972 at the river Athashi in the Kyrgyz Republic (formerly USSR), 79 m high, where a combined core wall has been constructed – one plastic and one grout (Fig. 14.42). Polyethylene strip, with a thickness of 0.6 mm, is placed in the upper 44 m of the dam, while in the lower part, in which there exist considerable pressures that the polyethylene strip cannot sustain, a grout core wall has been executed. The joint between them has been realized through a concrete grouting gallery, from which there have been constructed the grout curtain in the foundation, as well as grout core wall. The contact of the plastic core wall with steep banks of the canyon has been constructed by means of concrete equalizing and aligning slabs, anchored into the rock (Grishin et al., 1979; Rozanov, 1983).

From both sides (upstream and downstream) geotextile has been attached to the core wall, also with a thickness of 0.6 mm. The construction of the plastic core wall is between two layers of sand, with a maximum particle size of 5 mm.

The grout concrete wall, 22 m deep and 20 m wide, has been constructed of seven rows of boreholes, spaced 3.5 m apart, with distance between boreholes in a row of 1 m. Cement – bentonite solution has been used for grouting (cement 350-475 kg, bentonite 59-82 kg, and water 826-853 l per 1 m³ solution), injected under pressure of 0.5 to 3 MPa. Besides, in the centre of the concrete wall there have been constructed two additional boreholes, through which aluminium-silicate solution has been injected. Calculations have shown that by such unusual construction of the watertight element, by means of which the originally anticipated loam facing has been replaced, the cost of the dam has been significantly reduced.

Recently an internal geomembrane was installed as impervious core at the Gibe III cofferdam in Ethiopia, in the context of a hydraulic scheme with one of the largest hydropower plants in Africa. The project includes a 240 m high RCC dam and an upstream 50 m high rockfill cofferdam of around 500,000 m³, made of river gravel, basalt and trachyte. The construction of the cofferdam had to be completed during the short, six-month period of the dry season, when the average river flow is 200 m³/s. Of decisive importance in selecting a central geomembrane core were the following factors: short construction period, simplicity, lack of clay suitable for an impervious earth core and safety (Scuero and Vaschetti, 2011).

The impervious core, placed in a zigzag procedure during construction of the cofferdam embankment, consists of a flexible PVC geomembrane, sandwiched between two anti-puncture layers consisting of a high tenacity needle-punched geotextile. The geotextile is produced from 100% virgin polypropylene fibres, with a mass of 1200 g/m². Its function is to protect the geomembrane against possible damage during the placement of the upstream and downstream cofferdam fills. Two 50 cm thick sandy filter layers, with maximum grain size of 50 mm, were placed respectively at the
Figure 14.42 Atbashinskaya dam (Kyrgyz Republic) (after Grishin et al., 1979). (1) Rockfill; (2) rockfill into water; (3) gravel embankment; (4) grout concrete wall in the dam; (5) grout curtain in the foundation; (6) transition zone; (7) polyethylene concrete wall in sand; (8) rockfill; (9) alluvial deposit; (10) marbled limestone.

Figure 14.43 Typical cross-section of Gibe III upstream cofferdam (after Scuero and Vaschetti, 2011). (1) Basalt and trachyte; (2) selected rockfill; (3) clay cut-off; (4) filter layer; (5) pre-cofferdam made of gravel; (6) sandy clay; (7) internal zigzag waterproof element of geosynthetic; (8) bed rock below the river deposit; (9) relief wells.

upstream and downstream side to separate the waterproof element from the coarse embankment material.

The construction of the cofferdam was preceded by the construction of an approximately 20 m high pre-cofferdam, incorporated into the final cofferdam, to divert the Omo River into the diversion tunnels and to dry out the cofferdam foundation. In this way the cofferdam cut-off could be realized in clay (on which the geomembrane is encased), which waterproofs the riverbed alluvium and the shoulders colluvium.

The geomembrane was installed from the bottom cut-off up to the crest, in a zigzag pattern, so as to follow the step by step the construction of the embankment (Figs. 14.43 and 14.44). The waterproofing system thus creates a continuous impervious barrier running all along the longitudinal axis of the dam, from the bottom cut-off up to the crest. The first section of the cofferdam body is downstream directed and has a height of 6 m. The next sections follow one upstream and one downstream directed and have a constant height of 12 m.
The impervious element of the core was a flexible impervious 3.5 mm thick PVC geomembrane, resistant to deterioration under the alkali environment of damp concrete, and to degradation from organic and bacterial growth. The zigzag path of the waterproofing system was selected to provide sufficient material which can easily absorb any future deformation of the dam body caused by possible settlements. Furthermore the properties of the PVC geomembrane material, the anti-puncture properties of the geotextiles and the size of the aggregates composing the filter layers in contact with the waterproofing system should avoid any puncture or damage of the geomembrane.

The bottom anchorage was made by embedding the geomembrane in the 6 to 8 m deep clay cut-off and by backfilling with the same impervious earth material. At the two abutments, due to the difficulty of excavating the cut-off with the same depth due to the presence of surfacing rocks in the river bed, the geometry was slightly modified during construction, adapting the thickness of the clay layer below and above the geomembrane. The top anchorage of the geomembrane was made with steel anchor and plates fixed to the reinforced concrete crest wall.

When the 5 m of the temporary crest of the first section of the cofferdam body had been finished where the waterproofing system was to be installed, the surface was inspected, and deviations were corrected. The first anti-puncture geotextile was placed on the sandy filter layer. The geotextile was supplied in rolls 5.9 m wide and 42 m long, which were cut in sheets of sufficient length to cover the entire inclined slope and extend 2 m on the flat temporary crest. The geotextile sheets were placed vertically from the top to the bottom of the slope, with an overlap between adjacent sheets to allow joining by manual thermo-fusion seaming. At the toe of the first slope, the geotextile was placed in the cut-off. When the placement of anti-puncture geotextile was well advanced, the geomembrane, supplied in rolls 2.10 m wide and 12.5 m long, so that each roll could easily cover the entire inclined slope and extend 2 m on the flat temporary crest and 2 m more at the bottom of the slope, was placed on top of it. After
the placement geomembrane sheets were temporarily ballasted on the temporary crest with sand bags, and then completely unrolled down the slope. Adjoining sheets were joined by thermo-fusion seaming at the overlap. The seams, continuous for the entire length of the sheets, were executed by an automatic machine performing a double track seam, 100% tested with air in pressure.

In the areas where the geomembrane had been installed, welded and checked, a final joint inspection was carried out to verify that no defects were present before placement of the next section of the fill. Due to the fact that the cofferdam body was constructed by horizontal lifts of fill material, the placement of the protective geotextile on top of the PVC geomembrane had to be done with sheets placed horizontally from the bottom toward crest, following the placement of the lifts. The protective geotextile stops at top of the slope. The geomembrane placed on the temporary flat crest section 2 m wide was protected with stronger material to avoid major potential damages during the construction of upper part of the new section. After placing the second anti-puncture geotextile on top of the geomembrane, construction of the second section of the fill started, directed at the opposite side. The construction of the dam body proceeded by alternate sections, both upstream and downstream directed. The crest of each section is thus also the bottom of the section above it, and in this flat area the geomembrane lining the lower section is impermeably connected to the geomembrane lining the section above it. In the flat area at the crest of each section of the fill, the geomembrane lining the upper section overlaps the geomembrane lining of the section below it, over a width of about 2 m. Corresponding to this 2 m wide overlap area, the connection of the two sheets of geomembrane is made by means of a double track seam executed with automatic machine and tested as already described. The execution of this horizontal longitudinal connection seam, parallel to the axis of the dam, is made at the same time as the installation of the geomembrane sheets over the inclined slope of the upper section. Before the execution of this seam the protection placed to avoid damages on the geomembrane was removed, the area was cleaned, the integrity of the geomembrane was checked and, if needed, damages were repaired. The procedures described were repeated for each step of the construction of the dam body sections from the bottom of the cofferdam (elevation 670 m) up to the crest (elevation 720 m), where the upper edge of the geomembrane was mechanically fastened to the reinforced concrete crest wall (Scuero and Vaschetti, 2011).

14.3 STABILITY OF EARTH–ROCK DAMS WITH ASPHALTIC CONCRETE CORE

Regarding the conditions in which the static stability of earth–rock dams with concrete walls is accomplished, it is apparent that they are less favourable in comparison with dams with facing, because here the upstream shell is immersed in water. However, taking into consideration the materials of which rockfill dams are constructed as well as the fact that there is no pore pressure in any zone of the dam, it is clear that the static conditions of work of rockfill dams with concrete wall are more favourable in relation to the appropriate conditions existing in earth–rock dams. The author has performed detailed analyses of rockfill dams with asphaltic concrete watertight elements. For the applied methods and for the most important obtained results, there has already been
of the first twenty years following construction. Special jointing elements also enter into the analysis, by means of which it is possible to simulate cracks in the dam's body, as well as an assessment of their sizes (Hollingworth & Geringer, 1992; Schrader & Namikas, 1988; Zhu & Xu, 1995; Hansen & Forbes, 2012).

18.5 IMPROVING THE WATER-IMPERMEABILITY OF DAMS MADE OF ROLLER-COMPACTED CONCRETE

Providing the water-impermeability of the body of dams made of roller-compact concrete is a question to which, in the course of their more than 30-year development, there has been paid particular attention and, in that respect, time has brought permanent improvements and innovations. Dams of roller-compacted concrete, constructed according to the Japanese system in respect of water-impermeability, have turned out to be very successful. The wide wall of conventional concrete at the upstream face of the dam with the joints containing two seals, i.e. waterstops and a drainage opening, along with the joints cut into the roller-compact concrete, result in a final construction, which is very similar to the gravity dam of conventional concrete. However, this complex method of construction, although efficient, results in losing one of the most important advantages of the roller-compacted concrete—fast construction, which is possible only with simple structures. Because of that, a number of attempts and a lot of efforts have been made to find a method for achieving water-impermeability for dams made of roller-compact concrete, with, at the same time, the construction remaining as simple as possible and, thus, providing fast advancement during construction. Those attempts may be classified in the following way:

- **Execution of a monolithic thin upstream membrane of conventional concrete against formwork.** This method is the one most often used. The thickness of the membrane amounts 30 to 90 cm, and, in some cases, within the membrane there are embedded excitors of cracks (Middle Fork, Arabie, Grindstone Canyon) in order to keep the location of occurred cracks under control. This construction leads to certain difficulties during construction due to the need for a minimum of two types of concrete to be prepared simultaneously and also due to the potential danger of the occurrence of cracks in the membrane. But despite these difficulties, this method for improvement of the water-tightness of the RCC dams is the most popular.

- **Execution of a bonding mortar course between layers of roller-compacted concrete.** These mortar courses are built-in either when there is an upstream membrane of conventional concrete or independently, for preventing or setting back seepage between the layers of the roller-compacted concrete (Copperfield, Craigbourne). This concept is endorsed by the theory according to which the weakness of the dams of roller-compacted concrete, in relation to their water-impermeability, are precisely those joints between individual layers.

- **Execution of a geomembrane behind prefabricated concrete panels at the upstream face.** This method has been used for the Winchester dam, and has also been proposed for the Urugua dam (Argentina). The Galesville dam encompasses a membrane that has been formed by means of spraying of a rubber mass upon the poured concrete surface, but it has not succeeded in bridging cracks on the surface of the concrete.

- **Execution of a layer of asphalt mastic behind the prefabricated concrete panels at the upstream face.** This kind of construction of water-impermeable membrane has been used for the Kengkou dam in China.