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Production/ Publisher	Uitgeverij Educom BV

NGO-Adress:
Nederlandse Geotextielorganisatie (NGO)
Postbus 358
3840 AJ Harderwijk
Tel. 0031 (0)85 104 47 27

www.ngo.nl

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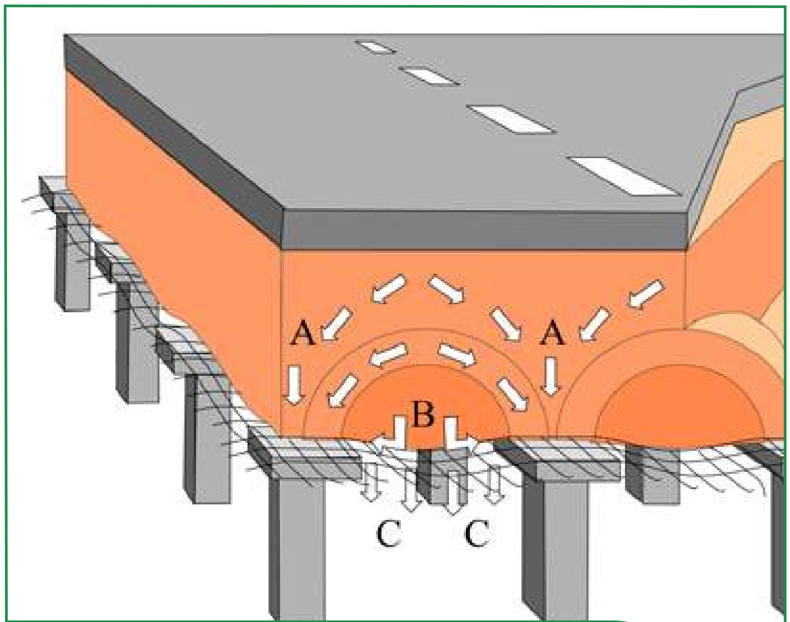

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The NGO welcomes you to the 10ICG and Baugrundtagung

Dear conference guest,

The Dutch chapter of the IGS (NGO) welcomes you to the 10ICG Conference on Geosynthetics at the ESTREL Convention Centre in Berlin. As neighbouring IGS Chapter to our German friends, we would like to take this opportunity to introduce you to the NGO, by means of this Conference Special edition of GeoArt.

The NGO (Nederland Geotextielen Organisatie) was founded in December 1983 and therefore even precedes the IGS. From the time the IGS was established the NGO became the Dutch Chapter. NGO has always been an active chapter of IGS. Our goal is to "Promote the responsible use of geosynthetics". We put a lot of effort onto PR. We have our website www.ngo.nl which is directly linked to the IGS site and our own magazine GeoKunst. "Kunst" means art or artificial. So the pun (GeoKunst / GeoArt) works in both languages. GeoKunst is published 4 times per year as an insert in GeoTechniek. 5000 copies of this magazine are distributed to just about everyone in The Netherlands and Flemish Belgium, who is interested in geotechnical engineering. We publish 2 full length articles in each edition. NGO organizes an annual NGO meeting in which we invite key note speakers from the Ministry of Public Works, engineering companies, contractors and producers of geosynthetics to present papers and to share their thoughts and experiences on developments in geosynthetics and the use of geosynthetics in civil engineering projects. Another typical activity we organize annually is our Geosynthetics Workshop. Each year a theme is selected: Dyke construction, piled embankments, slope stability, etc. The workshop consists of a number of presentations and always ends with a challenge. Teams are formed and each team is asked to build a scale model of their solution to the challenge, with the materials provided. The models are then "tested" and a much coveted prize is awarded to the winning team.

The Netherlands is a delta area and basin to the Rhine and the Muse rivers, which branch out in the Netherlands and finally flow into the North Sea near Rotterdam. Much of middle and west of the country has highly compressive soft soils (peat and clay) which makes life challenging when building infrastructure. The foundations of most buildings, bridges and viaducts in these areas are piled. Most roads and railroads are not, however the piled embankment is steadily increasing in popularity, as this has proved its self to be a highly effective construction method for building raised infrastructure on soft soils, especially as embankment to piled viaducts

***Figure 1** - The challenge 2014: Test to failure of a steep slope embankment*

or bridges. See the article by Tara van der Peet and Suzanne van Eekelen. The embankments themselves are often built with extremely steep slopes of up to 90 °, because of the chronic lack of space in the built up areas. In the steep slope constructions geogrids are used to reinforce the soil mass and make perpendicular slopes possible.

Another popular construction for embankments with steep slopes is the use of EPS blocks, which highly reduce the necessity of for soil improvement or deep road foundations.

The transition from an embankment on to a bridge or viaduct deck can be challenge if settlement is expected or due to thermally induced movement of the (concrete) bridge decks. In Jeroen Schrader and Arian de Bondt's article "Jointless Asphalt Pavements at Integral Bridges" Jeroen and Arian explain the development and the performance of apparently "jointless" continuous asphalt pavements over the transition from embankment to bridge. The joints are concealed within the asphalt construction and consist of a long approach slab, connected to the concrete construction by steel cables and layers of geogrid between the asphalt binder and wearing course.

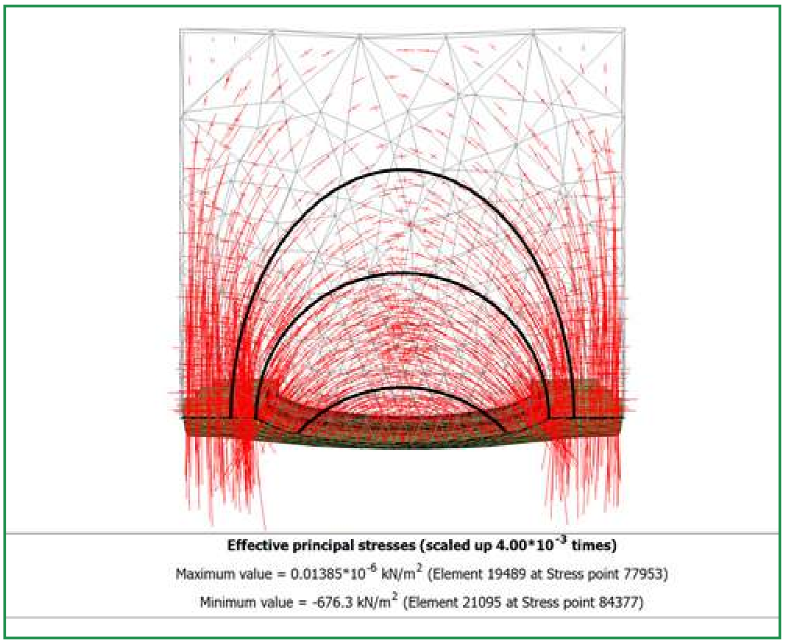

Figure 2 **-** *Concentric arches in a piled embankment*

Figure 3 **-** *Steep slope construction motorway A2, Utrecht 2011*

Figure 4 - *Viaduct with "jointless" asphalt pavement construction. Venlo 2011*

In most infrastructure crossings a raised embankment is used. But this is not always the case. In some cases it can be more economical or a better ecological solution to lower one of the structures, allowing it to cross beneath. In Rijk Gerritsen's article "Submerged Geomembrane Systems in Urban Areas" Rijk explains the principals and experiences in creating polders with a regulated water table, by applying a watertight geomembrane construction between sheet piled walls. The water level within the boundaries of the underpass can be maintained many meters lower that the surrounding area. Thus creating a "polder" with very limited width due to the sheet piling on both sides.

The Dutch have a long tradition of innovation in constructing infrastructure in difficult soil conditions and controlling and harnessing water. Dykes and levees have been built, maintained and improved for many centuries. At this point in time huge hydraulic works are being performed along the main Dutch rivers. After centuries of raising dykes to cope with increasing water levels in the rivers, the current program concentrates on widening the rivers and thereby creating a much greater cross sectional area (room) for the rivers when needed. The main point being, that it is much more effective to widen a river, than to increase the height of the dyke. Widening a river which is contained on both banks by dykes always involves building at least one new dyke. Dyke construction has proved to be an excellent opportunity to innovate using geosynthetics: grids, membranes, composites for erosion control, drainage, slope stability, impermeability, etc.
We hope you enjoy the conferences and your stay in Berlin, that you make good use of the opportunity to expand your network and that you return home with new ideas and enthusiasm for the many innovative and boundary breaking solutions to every day challenges, you will undoubtedly encounter during the technical program, discussions, lectures and presentations.

Figure 5 -*Sheet pile polder, Assen 2008*

Warm regards,
Shaun O'Hagan:
Editor in chief GeoKunst / Chairman of the NGO PR committee.
Colette Sloots: Text editor

Submerged geomembrane systems: Innovative polder-constructions in limited space

R.H. Gerritsen
Witteveen+Bos Engineering Consults, Deventer, The Netherlands

D.H. van Regteren
Genap BV. Geomembrane Systems, s'-Heerenberg, The Netherlands

R. Knulst
Dutch Directorate for public Works and Water Management, The Netherlands

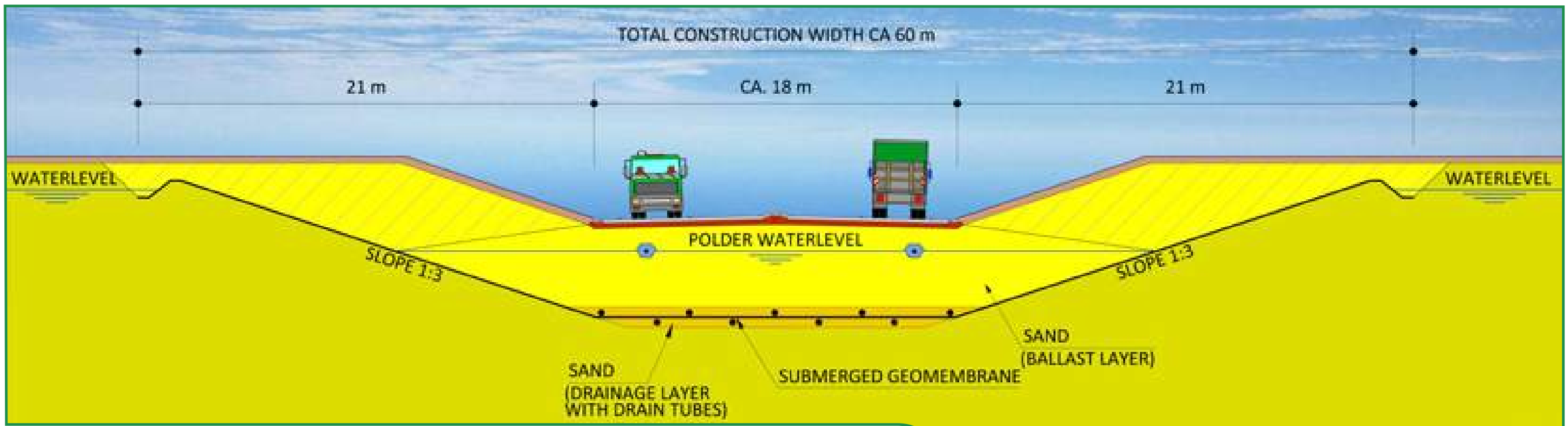

Figure 1 - *Submerged geomembrane with natural slopes with wide excavation dimensions*

1. INTRODUCTION

Geomembrane systems can be used as watertight artificial barriers in underground construction of roads, rail- and waterways. Examples of civil engineering applications are access roads (ramps) to tunnels, aqueducts and viaducts (pass ways, cross roads, sunken roads). The geomembrane system is used below the initial ground and groundwater level. High water tables up to the surface make it challenging to install the geomembrane system deep below the surface. In the Netherlands submerged geomembrane systems have been installed down to a depth of about 27 metres below the surface and groundwater table. The function of the geomembrane is to create an artificial impermeable barrier below the construction pit. After ballasting the geomembrane with sand, the groundwater level in the construction is set to a lower level than in the surroundings. The installation of a sealed underground construction basically creates an artificial polder, see figure 1.

The use of this building method is suitable for the typically Dutch soil and high groundwater circumstances, but will have also a high potential to delta areas abroad.

Figure 2 - *Geomembrane in open excavation before submerging (Aqueduct RW31 Langdeel, 2008)*

Abstract

Geomembrane systems can be used for civil engineering purposes in watertight sealing of underground constructions. Since the early 70's experience has been gained in The Netherlands with geomembrane systems in underground constructions for roads, rail- and waterways in open excavation pits. Due to the groundwater circumstances in the Dutch delta area most of the deep geomembrane systems are submerged, using PVC-p material (plasticized polyvinyl chloride). Submerging geomembranes (underwater installation) will require wide excavation dimensions. In urban areas most projects will have limited space. The dimensions of building pits can be limited in several ways, using some innovative design concepts. Construction concepts for use in limited space are the geomembrane U-polder and Sheet pile-polder. To prove the concepts several business cases and projects have been carried out. Geomembrane construction in limited space combines several known construction techniques like foundation works (sheet piles, anchoring), earth works, and submerging of geomembranes. The success of the concepts depends on a good understanding of design aspects, materials, risk-assessment and quality assurance during the building process. This article explains the concepts, trial testing and conditions for successful implementation of geomembrane systems with vertical boundaries in urban areas.

The easiest way to install geomembrane constructions is by means of open excavation pits. Combining constructions deep below the surface with underwater slopes of 1 : 3 (vertical : horizontal) the construction width can be extremely large, projects with dimensions of 250 meters width and over 800 metres length are no exception, see figure 2. This kind of spatial use will only be possible in low populated rural areas.

In urban areas the circumstances are more difficult because of nearby situated buildings, roads and pipelines. The dimensions of the building pit to install the geomembrane construction can be limited in several ways, using some innovative design concepts. Examples of the necessary construction widths are given in table 1. These widths are listed for a road construction with 4 lanes and a construction depth of 4 m-surface. This example illustrates that the innovative building method of a geomembrane sheet pile polder is highly competitive with the spatial use of a traditional building method with concrete and sheet piles. A geomembrane U-polder needs more width because of the inner support with soil slopes or retaining walls, but still the width is about factor ≥ 2 less in comparison to a geomembrane open excavation.

Table 1 - Example construction width (underground road with 4 lanes at 4 m-surface construction depth)

Design concept	Road construction width [m]	One side construction width [m]	Total construction width [m]
1. Concrete and sheet piles (traditional)	18	2	22
2. Natural polder (using soil layers, with natural slope)	18	12	42
3. Geomembrane open excavation	18	21	60
4. Geomembrane U-polder	18	6	30
5. Geomembrane Sheet pile polder	18	2	20

Remark - To the listed construction width it should be noted that some construction methods uses permanent or temporary anchor systems for wall support and economic design (method 1, 4 and 5). The spatial use of the underground anchor system is not included in the illustrated construction width.

2. DESIGN CONCEPTS

During the last decade a few design concepts have been developed in the Netherlands to limit the construction width for geomembrane installation below the water table. The main design concepts are:

1. Geomembrane U-polder
2. Geomembrane Sheet pile-polder

The elements of the concepts are clarified below. The concepts were originally developed by the Dutch directorate for public works and water management. To prove the concepts several trials and projects have been performed.

2.1 Considerations installation geomembrane

One advantage of submerging the geomembrane in an excavated building pit is that no dewatering of the building pit is necessary. Using this method will minimize the environmental impact and settlement risks to the urban area. Using PVC-P material with a higher unit weight than water, the geomembrane will sink due to its own weight. By using drainage pipes on the bottom of the pit below the geomembrane the water can be pumped from below to the top of the geomembrane. By circulating the groundwater the geomembrane will submerge gradually, see figure 3.

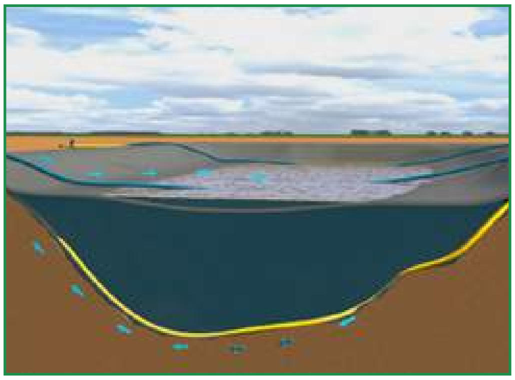

Figure 3 *- Principle of submerging the geomembrane with drainage systems and circulating water flow*

The material properties and quality assurance standards of PVC-P geomembrane used for submerging are listed in table 2. Normally the PVC-P geomembrane is prepared and manufactured off-site in a sealfactory (prefabrication). The advantage is that the circumstances in a factory are constant and ideal for sealing the geomembrane sheets. The PVC-P geomembrane will be composed to the exact 3-dimensional shape of the building pit, by welding 2 meter width geomembrane rolls together with hot-wedge or high frequency welding techniques. If feasible the PVC-P package will be transported as one sheet to the construction site. Prefabricated sheets of up to 5000 square meters have been produced off-site. If the area is too large to compose a single sheet, the prefabricated geomembrane sheets can be transported separately and welded together on site using hot-wedge welding techniques with testing channels. The channel welding can be tested by air pressure tests. The geomembrane package will be folded like a harmonica, so that the package can be

Figure 4 **-** *Special pontoon for storage and launching the geomembrane to the water level*

Table 2 - Material properties geomembrane Aquatex® - polyvinylchloride (PVC-P)				
Data	**Norms**	**Units**	**Thickness 1.0 mm**	**Thickness 1.3 mm**
Thickness		mm	1,0 - 1,1	1,3 - 1,43
Weight		gr/m²	± 1300	± 1690
Density	DIN 53479	gr/cm³	1,25 ± 0,03	1,25 ± 0,03
Tensile strength (L/T)	DIN 53455	N/mm²	≥ 18	≥ 18
Elongation at break (L/T)	DIN 53455	%	≥ 300	≥ 300
Tear resistance (L/T)	DIN 53363	N/mm	≥ 100	≥ 100
Dimensional stability (6 hrs at 80°)	DIN 53377	%	≤ 2	≤ 2

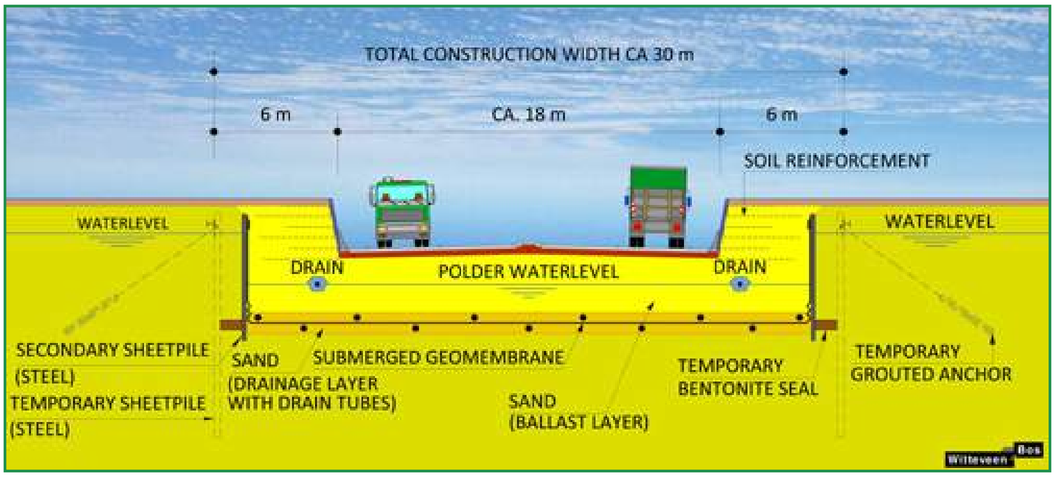

Figure 5 **-** *Design concept geomembrane U-Polder with temporary sheet piles*

unfolded easily. In view of the vertical boundaries on site the launching of the geomembrane package should however be prepared carefully.

In some cases a special pontoon was used, on which the geomembrane package was stored before it was launched to the water, see figure 4. Also a lot of experience has been gained with launching the geomembrane package directly from the edge using winches. The floating and positioning of the geomembrane is controlled by using buoys connected to the package. The submerging is started by circulating (pumping) water from the lower to the upper side of the geomembrane. This method is today common practice (CUR 221, 2009).

2.2 Concept U-polder

The U-polder is a geomembrane construction, using a minimum of structural elements like sheet pile walls and concrete. The geomembrane construction is installed in a U-form, with a horizontal base between two vertical limits (temporary sheet piles). The design concept of the U-polder is visualized in figure 5. The vertical stability is achieved by a ballast layer on the geomembrane. The horizontal stability during the construction phase is achieved by a primary sheet pile, if necessary supported by grouted anchors. These supported structures are temporary and can be removed after construction. After excavation a secondary front wall is installed, consisting of flat sheet pile profiles. The geomembrane is submerged and attached to the permanent front wall. A temporary bentonite seal is installed between the primary and secondary wall. After ballasting with sand the U-polder can be dewatered. The stability of the vertical geomembrane is controlled during the dewatering stage by lowering the water table between the primary and secondary wall. Using this method no build up of water pressures occurs behind the vertical geomembrane. The bentonite seal reduces flow rates in this temporary stage. For the final phase the horizontal stability of the geomembrane is achieved by the support of a retaining structure on the inner side of the geomembrane. From a geotechnical point of view this is the 'passive' ground pressure side of the geomembrane wall. The retaining structure in the U-polder can be constructed in several ways, e.g. a concrete gravity wall or reinforced soil structure (for example Terre Armee). The retaining structure will determine the visual perception of the U-polder in an important way.

The construction process starts with inserting the temporary sheet piles and excavation of the building pit to the installation depth of the geomembrane. In front of the sheet pile a second flat wall is installed, functioning as a lost formwork for the vertical geomembrane installation. By sealing the gap between the walls the water level between the two walls can be drained to

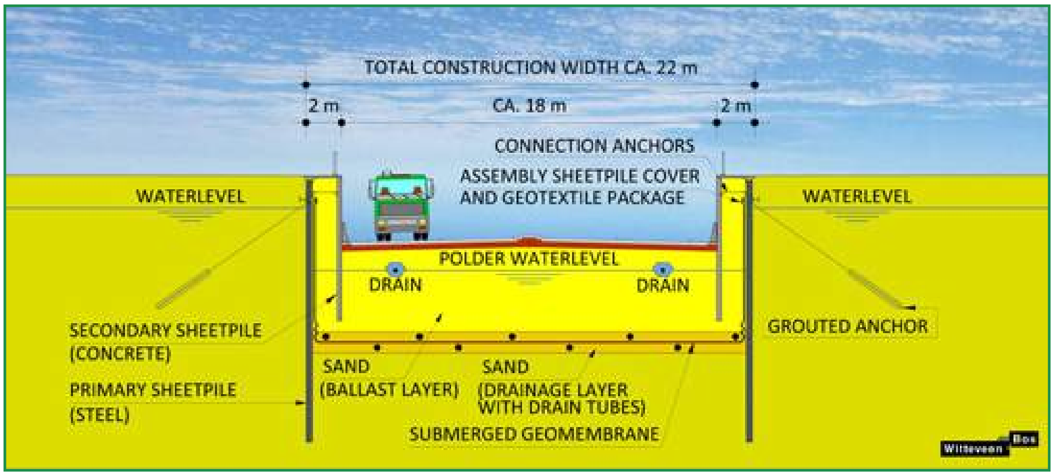

Figure 6 - *Design concept geomembrane sheet pile polder*

Figure 7 - *Sealing of the major geomembrane with hot-wedge channel welding techniques on site before submerging*

avoid high water pressures behind the upper part of the geomembrane. After ensuring the stability on the passive sides of the vertical geomembrane the temporary sheet pile walls can be removed.

2.3 Concept Sheet pile polder

The sheet pile polder is a geomembrane construction, which is installed between primary walls of sheet piles. The width dimensions of the underground construction are limited as much as possible by introducing a secondary front wall within the geomembrane construction. The design concept of the sheet pile polder is visualized in figure 6.

The primary walls can be supported by grouted anchors if desirable for an economic design. After wet excavation between the sheet piles, and protection measures in front of the sheet piles, the geomembrane is submerged (installed underwater) in a U-form. The vertical stability is maintained by ballasting with soil. The secondary wall is installed from each side after ballasting the bottom. The secondary front wall will also be the facing during service life. Considerations can be made about the type and facing of front wall, e.g. steel or concrete sheet piling. If necessary also an architectural facing can be introduced on the secondary wall.

The precondition to the design concept is to contain the structural forces on the secondary wall by the horizontal soil en water pressure. The forces in the secondary wall are transferred by means of a structural connection between the top of the two walls. Horizontal forces are transferred to the grouted anchors behind the secondary wall.

Particular care should be given to the protection of the geomembrane from accidents with fire or chemical exposure. However unlikely this is to occur during lifetime, the geomembrane must be protected from high temperatures. This can be solved by applying an appropriate front wall with sufficient protection. In case of major road constructions or routes with heavy chemical transport the front wall can be faced with a fire protective tile covering or by applying more distance between the primary and secondary walls. In high risk circumstances also a lot of attention should be given to the provisions, material specifications and detailing of the geotextiles.

3. TRIAL TESTING CONCEPTS

3.1 Trial test U-polder Ouddorp

The Dutch Directorate for public works and water management initiated the U-polder principle with the objective to save money in relation to basic and traditional building methods. In collaboration with three construction companies a test was performed in 1995 at the N57 in Ouddorp in the province of Zuid Holland. Main critical points that were researched: the underwater installation of the geomembrane against a vertical wall, the risk on damaging the geomembrane and the stability of the U-polder.

The positive outcome was that the installation of the geomembrane package underwater was feasible and the risk of damaging the geomembrane was controlled by using an additional steel support wall (Ruit et. al, 1995).

3.2 Trial test Sheet pile polder Voorburg

In 2001 a test was performed by the Dutch Directorate on Roads and waterworks on the installation of a sheet pile polder. The main objective of the test was to research two critical installation aspects:

1. Flattening out the trapezium profile of the constructive sheet piles by means of an economic and fairly simple covering construction.
2. Conserving the waterproof geomembrane during the installation of a secondary wall by means of woven and non-woven geotextiles.

During the field test several layouts were tested to cover the sheet pile cavities with more simple constructions than fully flat steel profiles with drainage holes. During the test several layouts were tested with covering the sheet pile cavities with reinforcement meshes, varying in rod diameter and size opening. The outcome of the test was published in October 2002 (Hemelop, 2002). The conclusions that were drawn were that the covering of sheet pile cavities is feasible with reinforcement meshes faced with an additional geotextiles at both sides of the geomembrane. A non-woven geotextile over the reinforcement rods minimizes the risk of damaging the geomembrane by sharp edges sufficiently and keeps the waterproof geomembrane from being pressed between the openings of the rods. A woven geotextile in on top of the geomembrane is used to protect the geomembrane against the high tensile forces due to the backfill and installation of the secondary wall. This secondary sheet pile wall was installed at varying distances of 1.0 to 0.35 m from the vertical back wall. Installation effects were measured with displacement transmitters on the woven geotextile. The major component of strain in the woven geotextile occurs during the backfill (2 to 2.5%). By installation of the secondary wall an additional strain occurs of 0.5% (De Vries, et. al, 2001).

4. PROJECT EXPERIENCES

4.1 Construction U-polder tunnel for low speed traffic Assen 2008

In Assen the access ramps of a tunnel for low speed traffic (cyclists) were constructed with the principle of the U-polder concept. The total construction length was about 150 metres, consisting of concrete tunnel segments in the middle over 30 metres and access ramps of 60

Figure 8 - *Geomembrane sheet pile polder in full construction (Assen, Peelo 2008)*

Figure 9 - *Final situation sheet pile polder with new city road (Assen Peelo, 2008)*

metres on both sides. In these ramps the geomembrane was submerged against a permanent anchored sheet pile wall (see figure 7). The gaps of the sheet pile walls were covered with reinforcement meshes and protective non-woven geotextiles. The final horizontal equilibrium as provided by applying an earth retaining slope and small retaining walls. The geomembranes are connected to the concrete tunnel segments with a watertight clamping construction at about 7 metres below the surface.

4.2 Construction Sheet pile polder main city road Assen 2008

The full concept of the Sheet pile-polder has successfully been implemented in project at Assen-city. The municipality of Assen took a political decision to double the main access city road in 2005. Doubling the main road should solve the problems with traffic jams for entering and exiting the city. For traffic engineering reasons the municipality choose a local underground construction of the main road, with an oval roundabout for local traffic on top. In a design contest the public tender was awarded to an innovative building concept making use of the Sheet pile polder. Using this concept it was feasible to submerge a geomembrane construction next to the main road, while the existing road remained open. This was a condition for all construction sequences.

The sheet pile polder in Assen was used for the two side access ramps to the underpass. The internal width of the underpass is about 18 meters. The total length of the underground construction was 300 meters, including 180 meters of sheet pile polder (2 x 90 meters). The middle section of 120 meters supporting the roundabout above was constructed in a traditional way, using sheet- and tension piles with underwater concrete. The connection of the geomembrane construction to the traditional concrete middle section was made by a clamp fixing. This fixing was prepared in a separated small building pit about 7-8 meters below the surface. After fixing the geomembrane section to the head wall in dry conditions, the temporary construction pit was inundated. Once the water levels were equal in the construction pits, the separation sheet pile was removed. The main package of geomembrane was unrolled along the length of the building pit. The geomembrane package with dimensions of 90 x 35 meter was moved into position between the sheet piles with attached air chambers, lifting hooks and winches (see figure 8). After positioning the main and sub geomembrane were sealed together with channel welding on a small pontoon. After connection the total geomembrane was submerged with a-pressure difference of 0.5 meter water.

During the designing and building process some construction parts of the sheet pile polder were optimized in comparison to the original concept (Meester, Gerritsen, 2009). For the front wall (secondary wall) concrete sheet piling was chosen, having the same appearance as the concrete structure in the middle. For installation purposes the distance between the primary and secondary wall was set to about 1.0 meter. Having this distance simplified the connection and the backfill between the walls. Also the risk of damaging the vertical geomembrane, while installing the front wall was reduced.

To simplify construction the contractor added vertical ground water pumping between the primary sheet pile and the geomembrane construction. By temporary drainage the horizontal water pressures were maintained at a safe level. After vertical ballasting the geomembrane with sand the polder water level was established. By working in this way, it was possible to install the secondary front wall in dry conditions. Also the connection and backfill between the walls was simplified by temporarily controlling the water pressures. After finalizing the concrete front wall, connections and backfill the drainage wells were removed. After a building period of about 1.5 years the underpass was successfully completed in 2008 (see figure 9).

4.3 Construction U-polder tunnel for low speed traffic Deventer 2009

In Deventer a tunnel for low speed traffic (cyclists) was constructed with the U-polder principle. The building method was slightly modified to the one which was used in Assen. One of the main differences was the submerging of the geomembrane directly in the sheet pile wall ca-

Figure 10 - *Geomembrane U-polder in full construction before submerging (Deventer 2009).*

vities. This method was chosen by the contractor and required additional wedges and welding to the major geomembrane. Also the positioning of the geomembrane in the building pit is critical. Using this method introduces risks of high tensile forces if the prefabricated geomembrane does not fit exactly the form of the sheet pile cavities. However the submerging of the geomembrane was successful and no damage occurred (see figure 10).

4.4 Construction Sheet pile polder underneath a railway crossing Schagen 2013

In order to enlarge the height between the existing road and the railway crossing the construction company decided to engineer a submerged road between sheet piles. Specific for the project was the very limited space due to the presence of a canal and cycle path to the sides of the existing road. After installation of the sheet piles the road was demolished and the building pit was excavated to the necessary depth. The sheet piles were flattened by means of reinforcement meshes. Innovative was the use of a waterproof geomembrane to which a protective non-woven geotextile was sealed. The geotextile was used to protect the geomembrane against puncture from the soil excavation and against punctures from the reinforcement meshes. After installing the combination of geomembrane and geotextile construction a secondary wall was installed into the back filled building pit. The geomembrane was temporarily fastened to the sheet pile and finally anchored in a trench beyond the definite sheet pile. A geo-electric leak detection survey was performed with the positive outcome of no leaks detected.

5. EVALUATION OF CONCEPTS

5.1 Comparison construction methods

The major differences between the concept of the sheet pile polder and U-polder are the elements used to maintain the horizontal equilibrium. For the sheet pile polder the equilibrium is maintained by the interaction between the primary and secondary sheet piling. These elements must function for the duration of the construction lifetime. For the U-polder the sheet piles can be used temporarily or permanently. The final equilibrium is provided by internal dead weight, e.g. the soil slope, reinforced soil or retaining structure. The construction methods are compared more detailly in table 3 and 4.

Table 3 - Comparison construction methods

Construction method	Sealing material	Stability vertical Direction	Stability horizontal direction
1. Concrete (traditional)	Concrete	Piles, concrete floor	Sheet piles, concrete wall
2. Natural polder (using impermeable soil layers)	Natural soil layers or artificial walls	Natural soil layers, artificial injection	Soil slopes, sheet piles, soil mix-walls, cement-bentonite walls
3. Geomembrane open excavation	Geomembrane	Backfilled soil	Backfilled soil
4. Geomembrane U-polder	Geomembrane	Backfilled soil	Retaining wall, reinforced soil
5. Geomembrane Sheet pile polder	Geomembrane	Backfilled soil	(anchored) sheet pile with front wall

Table 4 - Comparison construction methods

Construction method	Limited width	Experience	Sustainable building (CO2)	Costs
1. Concrete (traditional)	+++	+++	-	-
2. Natural polder (using soil layers)	+	+/-	+++	+++
3. Geomembrane open excavation	0	++	+++	+++
4. Geomembrane U-polder	++	+	++	++
5. Geomembrane Sheet pile polder	+++	+	++	+

5.2 Risk analysis

The success of geomembrane constructions in limited space is a result of the combination of several known construction techniques like foundation works (sheet piles, anchoring), ground works, and submerging of geomembranes. The success of the concepts will depend on a good understanding of design aspects, materials and on quality assurance during the building process. However risks can occur. Based on the construction sequence of the concepts the following main risks can be distinguished:

- stability elements in horizontal direction (sheet pile walls, anchoring, retaining structures, passive soil wedge, external water pressures);
- damage of the geomembrane during construction by contact to vertical elements (protection of sheet pile grooves, method of backfill, tensile forces);
- damage of the geomembrane by external water pressures versus the backfill levels, appropriate sealing and drainage materials);
- presence of environmental pollution in soil or groundwater (durability geomembrane);
- suitability of the in situ soil material for backfill on the geomembrane (admixture with cohesive or organic soil, presence of sharp stones or tree roots, possibility of reaching the required degree of compaction);
- damage of geomembrane during lifetime by a calamity with fire or aggressive liquids (maintenance).

5.3 Construction costs

The construction costs of innovative geomembrane constructions depend very much on the circumstances. The geotechnical circumstances such as soil type and groundwater levels are of major importance. The soil type will determine the re-usability of the excavated earth. The major cost components of an innovative geomembrane construction are given in table 5. For comparison also the major cost components are given of the traditional building method.

As a business case of the construction costs comparisons have been made between three types of building techniques: Two types of innovative geomembrane construction (with temporary or permanent sheet pile) compared with the traditional building technique (sheet piles, underwater concrete, tensile piles and structural concrete). The basis for the cost index graph is a road, constructed about four metres below the surface. In the graph the relations are given between the direct building costs (vertical axis) versus the width of the road construction

	Table 5 - Comparison major cost components		
	Innovative geomembrane construction		**Traditional building method (underwater concrete and tension piles)**
1	Installation (temporary) sheet piles and anchor systems (more heavy dimensi-ons)	1	Installation (temporary) sheet piles and anchor systems
2	Soil excavation (deeper)	2	Soil excavation
3	Submerging geomembrane construction	3	Installation tension piles
4	Consideration of external Quality Assurance (QA)	4	Casting underwater concrete floor
5	Consideration of leakage detection method	5	Casting structural concrete floor and walls
6	Re-use of excavated soil or supply good quality backfill sand		

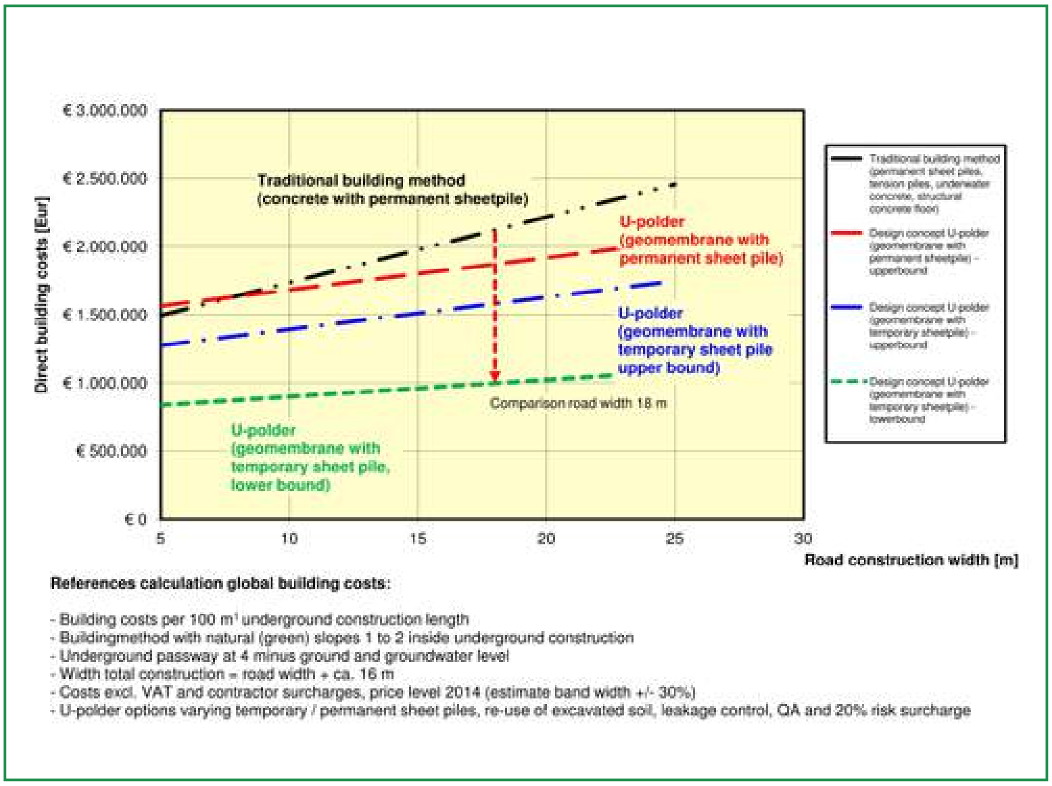

Figure 11 **-** *Relation direct building costs design concepts with increasing road construction width*

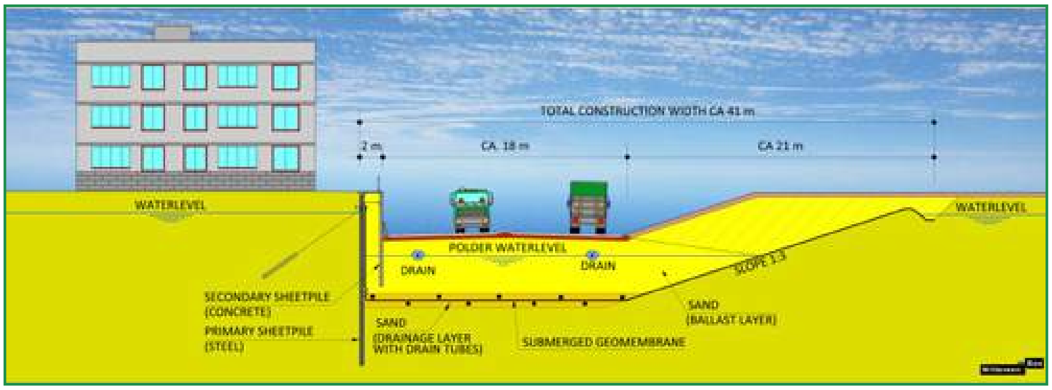

Figure 12 **-** *Application of a vertical limiting geomembrane on one side due to specific project circumstances*

(Gerritsen, 2012). The costs are capitalized to a construction length of 100 meter with green slopes to both sides. For the construction costs unit prizes are used with price level 2014. The costs of the concrete construction include sheet piles, underwater concrete, tensioning piles and a construction floor. The costs of the innovative geomembrane systems include costs of temporary or permanent sheet piles, extra excavations, geomembrane and ballast sand. The band width of the cost calculations is estimated at +/- 30%.

The starting costs of narrow road constructions (set on 5 meter) are high (> 0.8 mEUR per 100 meter). This is related to the high initial costs of installing sheet piles and related works to the vertical boundaries. If the road construction is wider the costs gradually increase. The costs per square meter decrease because of the relatively low price of the geomembrane sealing. Excavation works and transportation of soil are one of the major cost items of the geomembrane methods, accounting for thirty-five percent of the total building cost. The maximum of road width in the graph is set to 25 m, corresponding to a width of a 6 lane motorway.

From figure 11 we can conclude that innovative geomembrane systems are economically seen, a good alternative for almost all circumstances. Only in the case of a very narrow road construction (< 7 m), a permanent sheet pile wall and new backfill material, the direct building cost will be in the neighbourhood of a traditional building technique. As upper bound circumstances the calculation of the red line is including supply of new backfill sand, leakage detection, external QA and a risk ratio of 20%. In case the sheet pile wall is temporary the costs decrease significantly (blue line). In case of lower bound circumstances the cost relation is given by the green line. Taken a situation with an underground road construction of 18 m width the price ratio of a geomembrane concept is to be a percentage of 10% to 50% cheaper than a traditional building technique with concrete.

The most suitable locations are locations with sand and high water tables. However if cohesive or organic soil layers are present (clay and peat), the application of geomembranes can still be economical. The reuse of weak cohesive soil layers below the water table within the construction should be strongly dissuaded, as the soil matrix will be lost and compaction is difficult. If the soil in not suitable, then new sand can be transported to site for backfill above the

***Figure 13** -Building pit with submerging a geomembrane in a two phase working sequence (Hilversum, 2002)*

geomembrane. Even in case of backfill with new sand this should be economical in most cases, compared to traditional building techniques.

6. SPECIFIC APPLICATIONS

The construction width can be reduced on both sides by using the U-polder or Sheet pile polder concepts. However the width limit can also been applied to one side to solve a specific problem with the space use (see figure 12). With custom made design considerations the building costs can be reduced further. In the case of specific local bottlenecks the width limits can be applied over a restricted length. Crossing the bottleneck the vertical installation can gradually transfer to a standard geomembrane keel construction. Transferring the position from vertical to side slopes much attention should be given to the gradual curves, avoiding tension stresses in the geomembrane construction.

Previously the most innovative geomembrane constructions have been applied to civil road projects. However there is a lot of potential for application of innovative geomembrane systems to other structures. The submerging of geomembranes can also be applied in building pits for underground parking garages or large basements under buildings. In the Netherlands there are several references of using geomembranes as a temporary sealing within a building pit, e.g. projects in Kampen and Hilversum (see figure 13). Also submerged geomembranes can be used as temporary casting basins for large underground tanks or basins, like water purification plants. Besides temporary applications for water sealing during the construction phase, submerged geomembranes within limited space can also be used permanently. For permanent applications a lot of attention should be paid to detailing of the fixing system, working method and quality of the backfill material, durability of the geomembrane material, and drainage facilities (in case of unexpected leakage).

7. CONCLUSION

The width dimensions of constructions can be limited in several ways, using some innovative design concepts. The geomembrane U-polder and Sheet pile-polder are concepts which limit construction width. With the concept of the Sheet pile-polder the width dimensions can be equal to a traditional building technique with concrete. For the concept of U-polder the width dimension is 5-10 meters more per boundary. Compared with submerged geomembrane systems in an open excavation, the reduction of width dimensions are remarkable. Reducing the dimensions the use of geomembranes becomes a potential building method in complex circumstances like urban areas. Summary of the advantages:

- Minimizing building costs.
- Reduction of noise during foundation works.
- Minimize spatial use construction width.
- Rapid building time.
- No necessity for a large scale ground water dewatering.
- Implementing an integral approach will lead to safe and durable final solutions

The most suitable locations are locations with sand and high water tables. However, if cohesive or organic soil layers are present (clay and peat), the application of geomembranes can still be economically attractive. In comparison to traditional concrete building methods the benefits of geomembrane concepts give a considerable reduction of direct building costs and better valuation to sustainable building. Reducing building costs can change the environmental town and country building preference from infrastructure on ground level to sunken or underground infrastructure. The concepts have been proved in several trials and the full concept or variant components have been successfully implemented in several projects. Based on the typical circumstances the concepts of innovative polder-constructions will have a high potential to populated delta areas abroad.

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- Figure 1, 5, 6 and 12: Witteveen+Bos, design concepts, Otto Kuypers, 2014
- Figure 8, 9, 10, 11: Witteveen+Bos
- Figure 4: Dutch Directorate for public Works and Water Management
- Figure 2, 3, 7 en 13: Genap Geomembrane Systems

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CASE STUDY : Ref 310

Car parking at the Gibside Estate, Newcastle Upon Tyne, UK, 2012

Tensar MSL™ (mechanically stabilised layer) incorporating TriAx® geogrid used to enable construction over unexpectedly soft soils

Tensar MSL enables initial access over low strength soil

BENEFITS TO CLIENT

By adopting a Tensar MSL™ (mechanically stabilised layer), construction work was able to continue without delay and with reduced costs despite the discovery of extremely soft ground conditions.

THE PROBLEM

Unexpected low strength ground conditions discovered on site presented the project team with the potential for additional excavation and delayed construction which would result in increased costs. Even gaining access to the area was proving difficult as CBR values less than 0.5% were encountered across the site.

THE SOLUTION

A Tensar MSL incorporating layers of Tensar TriAx geogrid was installed to allow safe and stable access onto the soft ground with further assessment made to provide a stabilised foundation to the final car park construction.

PROJECT DESCRIPTION

Work had commenced on constructing an additional car park at the National Trusts' picturesque Gibside Estate on the outskirts of Newcastle-upon-Tyne when site operations had uncovered areas of existing formation soil with significantly lower strength soils than had been anticipated. Formation CBR values as low as 0.4% were measured in the exposed soils.

Following the discovery of these extreme ground conditions, Tensar International was approached by the project team to provide proposals to enable construction to continue and avoid unnecessary delay.

Tensar's engineers proposed a Tensar MSL to provide safe and stable access over extremely soft soil conditions and ultimately to provide a stabilised foundation for the car park when construction had been completed. A locally sourced recycled granular fill was used along with layers of Tensar TriAx geogrid to stabilise the granular layers. The Tensar proposal reduced the amount of additional excavation into the soft soils as well as minimising the amount of imported stone required in the construction phase. As a result, a cost saving of around 10% was achieved compared to that for the original construction depths envisaged.

Despite operating close to the low strength formation soils, construction operatives could work in safety and the inclusion of TriAx geogrids stabilised the construction to allow vehicles to traffic the area and maintain the construction programme.

Ben Johnson, Site Agent for contractor Owen Pugh, said that he approached Tensar *"…for a quick response and suitable design to reduce the cost of imported materials, and increase the stability of the car parking construction, to ensure most cost effective solution for the client."*

Jointless asphalt pavements at integral bridges

ir. J.G.F. Schrader
Ooms Civiel bv,
the Netherlands

dr. Ir. A.H. de Bondt
Ooms Civiel bv,
the Netherlands

Figure 1 - *Overview of situation of the bridge at Son, late 1999*

INTRODUCTION

The transition between the pavement on a bridge deck and the pavement laying on the natural soil has been a problem for a long time. If the asphalt pavement is simply paved without measures onto the bridge deck, one can expect after a few or even during one severe winter, that wide cracks become visible at the bridge end. This is due to the large strains which are generated in the asphalt concrete layers, especially at its bottom "fibre", de Bondt (1999).

Given the problem described above, several types of joints have been developed and applied over the past years. However, these joints have in common that their lifetime is short and difficult to assess. This means that quite rapidly and often unexpectedly (costly) maintenance, in the form of replacement is needed, Maijenburg (2000). As an example: an asphalt plug joint has an expected life of three years on an average motorway in the Netherlands. In the situation such as in the Netherlands, where the current motorway system is already loaded beyond its capacity, closing lanes for joint maintenance causes a lot of disturbance and is unacceptable from the user point of view.

It is clear that long lasting jointless asphalt pavements at bridge ends should be developed. More specific, the wearing course layer should be a continuous layer that complies with the maintenance regime of the adjacent asphalt concrete (mostly based on ravelling, rutting or lack of skid resistance and which normally varies between about 10 and 15 years), while the underlying binder asphalt concrete layer should be free from cracking for at least 50 years to comply with the maintenance interval of the bridge itself (often planned every 50 years). This means that the renewed wearing course does not have to be applied on a (severely) cracked layer, and that the chance of crack propagation from the binder layer will be eliminated.

CHALLENGE

The integral bridge used to develop the jointless asphalt pavement is located in the A50 motorway at Son (near Eindhoven) in the Netherlands. Figure 1 shows a photo (from late 1999) of the 70 m long bridge at Son (crossing the Wilhelmina-canal), in the phase before earth construction work of the road had been started. The construction of this bridge was finished in 1997.

The area in which the road was constructed, has good supporting sub grade conditions; at least from the Dutch point of view, since the soil consists of sand. However, given the fact that an embankment needed to be built, so-called approach slabs were utilized. The function of an approach slab is to create a smooth gradual pavement surface profile in case of settlements; in other words to avoid sudden bumps when "hitting" the bridge deck pavement. For this reason, these slabs are placed at an angle (see figure 2). At the bridge in Son this angle was 2.6 °.

The connection between the bridge deck itself and the approach slab is via steel cables, which are embedded in such a way that only rotations are possible (in case of settlements). This configuration implies that the approach slab will be subjected to the thermal expansion and contraction process of the bridge. Given the length of the bridge (70 m) and the length of the approach slabs on both sides (each side 5 m), it is clear that the amplitude of the thermal movements (summer/winter cycle) is quite large.

From the foregoing it can be concluded that if the wearing course layer should be a continuous layer with a maintenance interval for the criterion cracking (caused by the thermally induced bridge movement), which is at least similar to the maintenance interval of the bridge itself (a period of 50 years), a complicated design problem would arise. All in all, the challenge that the Research & Development department of Ooms Civiel bv took, was defined as follows:

"Develop (design) a cost-effective jointless pavement near a bridge end (including the preparation of tender specifications), which can sustain 2 mm daily movement (day/night) and 20 mm seasonal movement (summer/winter); this for a period of 50 years (under Dutch climatic conditions)"

Abstract

Bridge decks expand and contract during a year due to temperature variations, as any other "non-restrained" structure. The amplitude of this movement depends on the type of bridge, its length and the climatic circumstances. There are several different types of bridge structures and an integral bridge is one of them. The most characteristic aspect of the integral bridge is the fact that the (continuous) concrete bridge deck only rests on steel bearing piles, concrete columns or a concrete wall. It is clear that given the relatively low rotational stiffness of these supports, as compared to the bridge "power", a considerable thermal movement at the bridge ends needs to be taken into account, when designing the transition to the road pavement. This paper describes the development of a specific method to construct this transition without a visible and noticeable joint at the asphalt surface, and subsequently 11 years of field experience of the method, at several locations across the Netherlands.

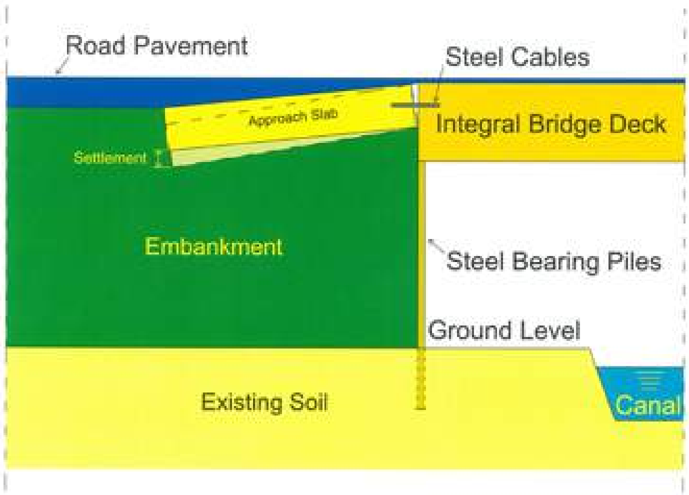

Figure 2 ***- Explanation of the function of an approach slab***

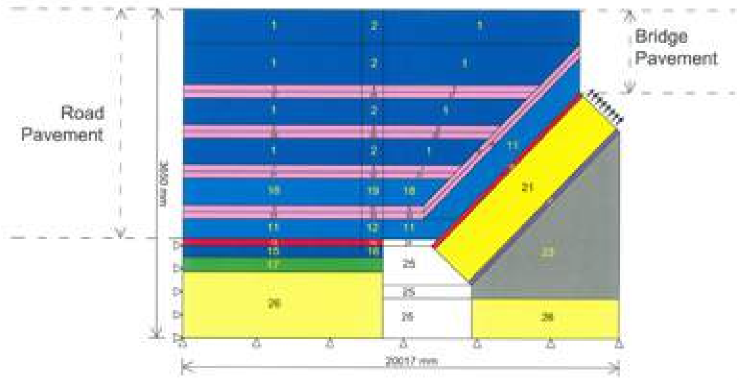

Figure 3 ***- Sketch of the super-element configuration (not to scale!)***

Table 1 - Explanation of super-elements

Material / Interface Number	Description of modelled parts of the integral viaduct
1/2/11/12/15/16/18/19	Asphalt concrete layers
3/4/5/6/7/8/9/10/27/28/29/30/31/32/33/34/35/36	Pavement layer interfaces (bond)
13/14/20	Stress-relieving system
17	Unbound granular base course
21	Approach slab (PCC)
22	Dry friction simulation
23	Cement stabilized sand
24/25	Air (simulation of no contact)
26	Sand sub-base course
37/38/39/40	Asphalt reinforcement

FINITE ELEMENT ANALYSES AND ENGINEERING

It is obvious that this goal could not be reached by performing rather simple mechanical analyses and that given the complex geometry the solution had to be found in using finite element modelling. The development work started by preparing a three-dimensional finite element mesh of a composition of the pavement structure which was at that time thought to be adequate. This means refining the mesh size at the locations that were thought to be critical. The program CAPA-3D, Scarpas and Karsbergen (1999) was used for the analyses. In figure 3 the main layers of the final mesh are shown and Table 1 gives an explanation of the material / interface numbers given in figure 3 (note that the reinforcement elements are not shown; they can be applied in between two pavement layer interfaces).

The complete mesh is subdivided into 5265 cubic elements and 1250 interface elements. A detail of the mesh, focussed on the critical locations, is presented in figure 4. The angle under which the approach slab (displayed in yellow) is placed, can be recognized, as well as the refinement of the elements around the transition from the approach slab to the unbound granular base course (displayed in grey-white).

Figure 5 shows the exaggerated deformation at the critical locations. From figure 5 it is obvious that the critical section is located in the asphalt layers on top of the transition between approach slab and unbound granular base.

To get proper material properties to input into the FEM-model extensive laboratory testing had to be performed, such as the determination of the different interface shear stiffnesses and the development of an extremely ductile (but still stable enough) asphalt type called Thermifalt.

The analyses showed the necessity of applying 4 layers of glass fibre reinforcement GlasGrid® 8501 (more specifically GridSeal®) in between the asphalt layers, and the need to apply

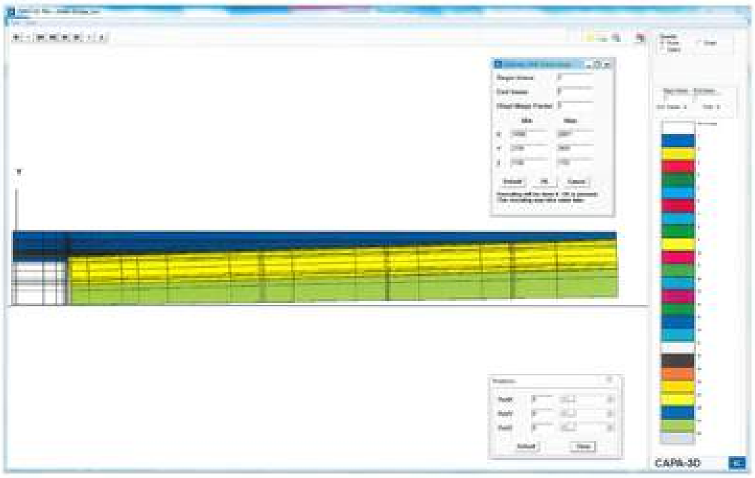

Figure 4 - *Detail mesh, focussed on the important locations*

Figure 5 - *Deformations around critical location (exaggerated)*

Figure 6 - *Installing the invisible joint system*

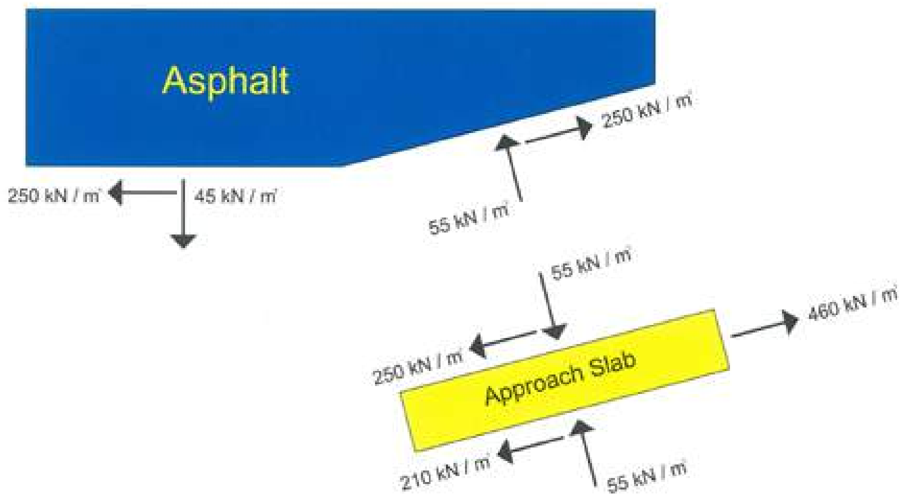

Figure 7 - *Sketch of equilibrium of forces (free body diagram)*

Sealoflex® polymer modified asphalt concrete in between the wearing course and the Thermifalt asphalt layers described earlier, de Bondt and Schrader (2001). Figure 6 displays the installation of the reinforcement.

Evaluating the analyses results, it became clear that the summer/winter case was more damaging than the day/night case. In order to illustrate the mechanisms which occur, an example of the forces which are acting on the approach slab and the asphalt is given in figure 7. Values are given per meter width of the bridge (note that these are rounded off).

It can be seen that with the current configuration (and input data) roughly half the restraint force is generated by the jointless asphalt pavement and roughly half the restraint force by friction between the approach slab and the cement stabilized sand underneath. An interesting aspect is that the generated force in the steel cables, which connect the approach slab and the bridge, was higher than initially expected by the bridge engineers. This had led to some design changes of the integral bridge in between the bridge and the approach slab, caused by the presence of the invisible joint system. Figure 8 presents a sketch of the forces along the critical cross-section in the asphalt (for a certain scenario).

It can be deduced from figure 8 that the asphalt takes 50 % of the generated force in the cross-section and the reinforcement 50 %. The analyses also showed that, in the long run, cracking of the bottom asphalt layer will occur. After this cracking, the forces will shift into 40 % in the asphalt and 60 % in the reinforcement and no further cracking of the asphalt layers will occur.

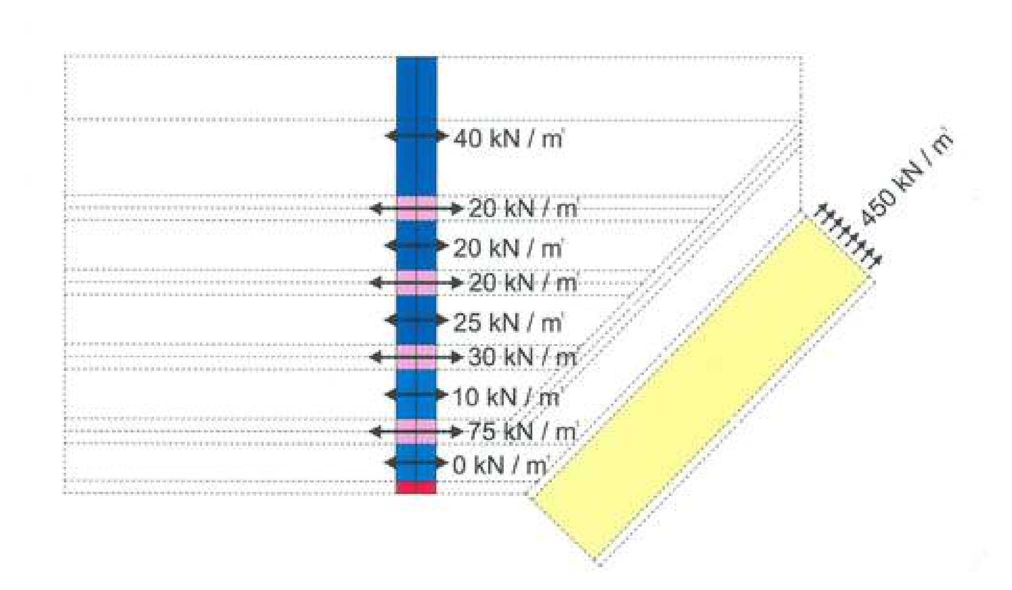

Figure 8 - *Detailed sketch of forces along critical cross-section*

Measuring instruments were built into the asphalt layers of KW01 in the A50 motorway to check whether the concept's performance is in line with the requirements and to verify the complex computations, see figure 9. These instruments generate data on the movement of the bridge deck and the strains in the asphalt layers (both as a function of temperature), see figure 10. The data has also been used to optimise the concept for other situations.

THE NEXT PHASE: A STANDARDIZED SOLUTION

The analyses have been performed together with the Engineering Office on Bridges and Tunnels of the Dutch Road Administration ("Bouwdienst Rijkwaterstaat") and finally the first "invisible joint system" was constructed at 3 locations on motorway A50 in 2003.

The Dutch Road Administration was so confident about the analytical solution and the actual construction in the field that in 2008 they upgraded the invisible joint system to the standard "joint" solution for integral bridges. For this, it was necessary to determine the solutions for integral bridges with shorter and longer bridge decks. This resulted in a table where a given temperature related bridge deck movement is translated into a specific number of asphalt layers (2 different types: Sealoflex and Thermifalt) and a specific number of reinforcement layers to be applied. Also the presence of an asymmetric viaduct or non-perpendicular joints were taken into account within the standard solution.

FIELD EXPERIENCE

Table 2 shows an overview of the locations where the invisible joint system has been constructed since 2003. Invisible joint systems have performed as expected, even after 3 extreme winters (according to Dutch circumstances) during the last decade. Meanwhile the wearing course of KW01 at the A50 has been replaced (because of ravelling) without any problems or damage to the invisible joint. The behaviour of the invisible joint systems at the three viaducts in the A50 has been monitored visually since their construction as well as the traditional joints of other viaducts in the A50. After 11 years, wide cracks are visible at the surface course at all joint systems other than the invisible joint system. During this period some other systems even needed to be repaired more than once.

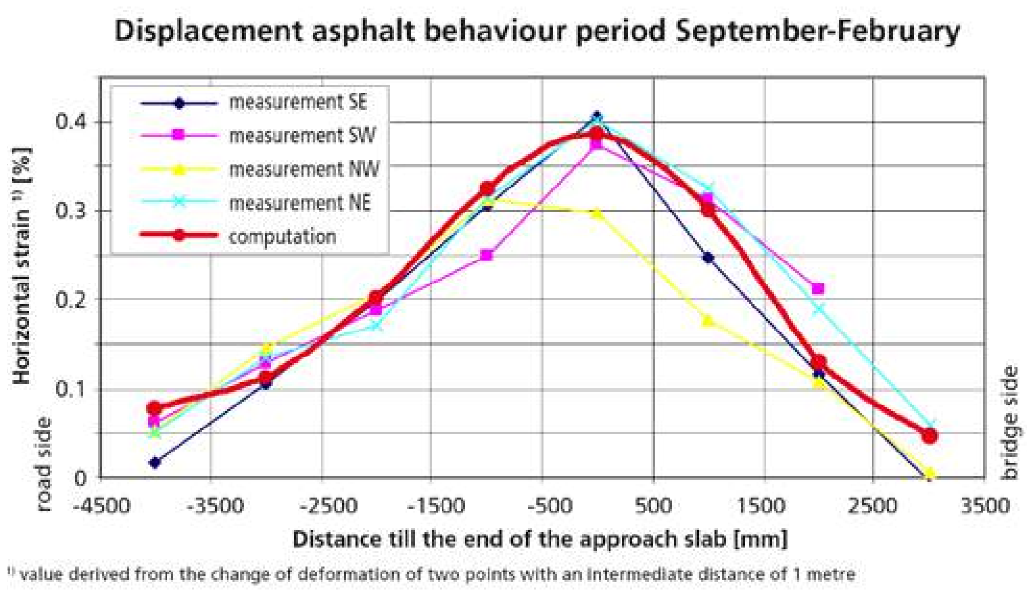

Figure 9 & 10 - *Measuring instruments & Generated data*

Table 2 - Field experience invisible joint system

Year of construction	Road name	Location
2003	A50	KW01 – Eindhoven-Oss
		KW26 – Eindhoven-Oss
		KW29 – Eindhoven-Oss
2007	A73	KW38 - A73-Zuid
		KW39 - A73-Zuid
		KW42 - A73-Zuid
		KW43 - A73-Zuid
2008-2009	A2	KW15 – Randweg Eindhoven
	A50-A58	KW41 – Knooppunt Ekkersrijt
2011	N247	Schardam, Beetskoogkade-Dorpsweg
	A74	KW04 - near Venlo
	A4	KW Dwarswetering, near Hoogmade
2011-2012	A12	KW09 - Poort van Bunnik
		KW17 - Poort van Bunnik
		KW21 - Poort van Bunnik
		KW25 - Poort van Bunnik
		KW28 - Poort van Bunnik
2014	N279	Integral bridge N279, near ´s-Hertogenbosch
	A50	KW11 – Eindhoven-Oss
		KW14 – Eindhoven-Oss
		KW16 – Eindhoven-Oss

Figure 11 ***- KW04 - near Venlo on the A74, during construction in 2011***

KW04 (an integral viaduct) near Venlo on the A74 motorway has to be mentioned separately, as the angle between the bridge and the road was extremely sharp (about 18 degrees), see figure 11. This angle was far beyond (below) the original limits of the invisible joint system, so complementary analyses had to be performed to solve this challenging problem. It became clear that under the wearing course 6 layers of highly modified asphalt concrete combined with 6 layers of glass fibre reinforcement were required to "absorb" the expected 30 mm summer-winter movement.

CONCLUSION

Based on the work described above, it can be concluded that via adequately detailed three-dimensional finite elements modelling, in combination with sufficient material testing and the use of high quality materials, it has been possible to develop durable (long-lasting) jointless asphalt pavement structures even for bridge ends which move 30 mm during a summer/winter cycle. This conclusion is justified by field experience over the past 11 years.

ACKNOWLEDGEMENT

For their stimulating discussions during the development of the invisible joint system and their willingness to innovate, Wim de Bruijn (now retired) and Frans van Gestel of the Engineering Office on Bridges and Tunnels of the Dutch Road Administration ("Bouwdienst Rijkwaterstaat") are highly acknowledged. This also applies to the efforts of Joep Thijs (Dutch Road Administration) to optimize the system for implementation on the A73, as well as the efforts of Tim Janssen (Royal HaskoningDHV) to extend the limits of the invisible joint system to create a solution for the A74. Finally Wouter van Bijsterveld has to be acknowledged for his contribution during his employment at Ooms Civiel.

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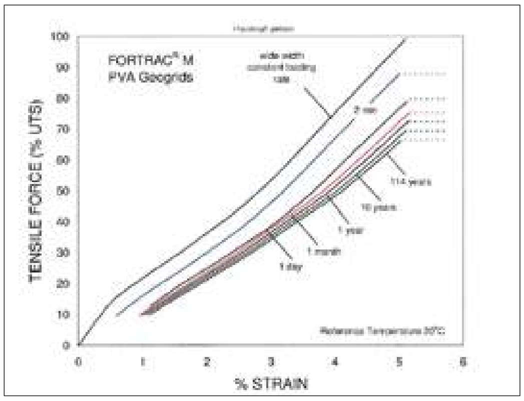

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3D numerical analysis of basal reinforced piled embankments

ir. T.C. van der Peet
Witteveen & Bos, The Netherlands

ir. S.J.M. van Eekelen
Deltares, The Netherlands

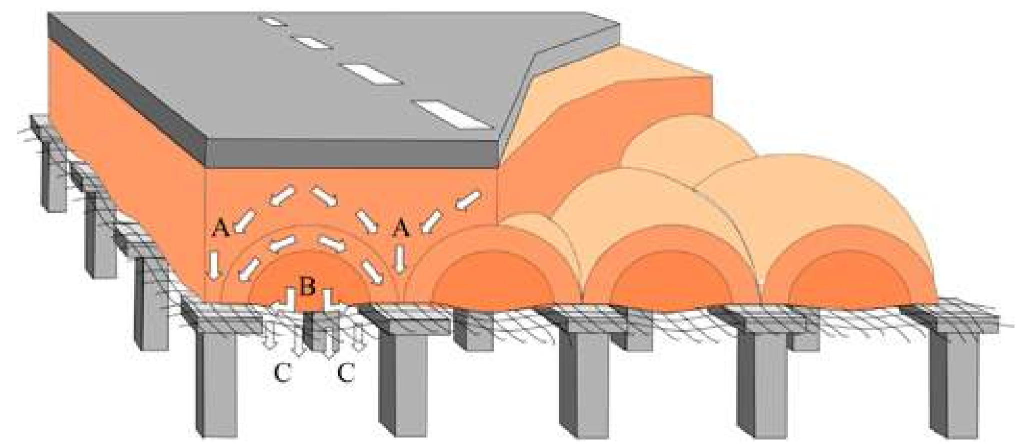

Figure 1 **-** *Load parts defined in a basal reinforced piled embankment: Arching A, GR force B and subsoil support C*

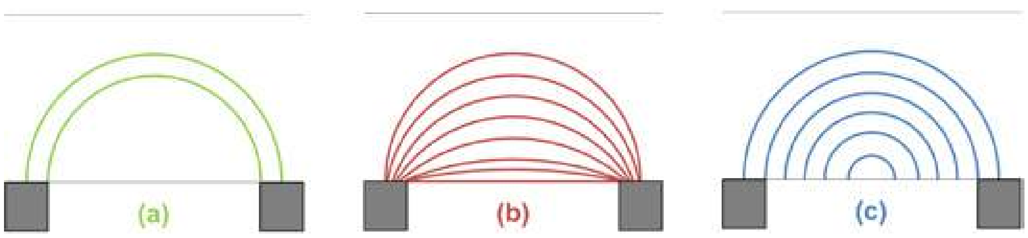

Figure 2 **-** *2D schematization of the (a) Hewlett and Randolph model, (b) Zaeske model and (c) Concentric Arches model. 3D aspects of the models are not included*

Piled embankments are used for road or railroad construction. They can speed up construction time in soft soil environments and reduce stability and deformation risks. A piled embankment consists of a field of piles on which an embankment of granular material is built. Basal reinforcement can be applied by placing geosynthetic reinforcement (GR) in the base of the embankment. The relatively stiff piles attract load from the embankment by means of arching (load part A). This lateral transport of load uses the friction of the granular fill. The load on the GR is partly carried to the piles through a tensile force in the GR (load part B), and partly carried by the subsoil (load part C). The load parts are shown in Figure 1.

ANALYTICAL MODELS

Described below are three limit equilibrium models. In such arching models, an imaginary stress-arch is assumed to appear above the void between the piles. The equilibrium of the arch, which is assumed to be in limit state, leads to the load distribution. The major principal stress is assumed to follow the arch shape. A 2D schematization of the three models can be found in Figure 2. It should however be noted that the models all describe a 3D situation and 3D effects that are included in the model but are not visible in this figure.

The model of Hewlett and Randolph (1988) is adopted in the French ASIRI guideline (2012) and suggested in BS8006 (2010) as an alternative model. The model is based on tests in which no GR was used. It assumes that one arch forms between the piles, limited by two concentric semi-circular borders.

The model of Zaeske (2001) is adopted in the German EBGEO (2010) and the current Dutch CUR226 (2010) guidelines. It assumes the formation of multiple arches, based on the pile caps of diagonally neighbouring piles. The boundaries of the arches are semi-circular, but non-concentric, leading to wedges that are thicker in the crown than in the toe.

The Concentric Arches model (Van Eekelen et al., 2013) uses a system with both 3D hemispheres and 2D arches. The hemispheres form above the part of the GR between the corner points of four piles, the GR square. They exert some load on the GR and transfer the rest to the arches, which form above the parts that lie directly between two neighbouring piles, the GR strips. The arches then exert some load on the GR and transfer the rest to the piles. Both hemispheres and arches have semi-circular, concentric boundaries, which implies some of them are based on the GR. The Concentric Arches model will be included in the modified version of CUR226 (2015).

NUMERICAL MODEL

The numerical model was drawn up in the computer program Plaxis 3D (version 2013) and uses the finite element method. The geometry can be seen in Figure 3. It does not include pile caps and an intermediate sand layer between GR and piles, although these are often applied, to simplify the calculation. The model incorporates two fields, two half piles and four quarter piles. The pile has a width realistic for a pile cap, b = 0.75 m. The centre-to-centre distance was chosen $s_x = s_y = 2.25$ m. Furthermore, in the basic situation, GR Stiffness J = 1500 kN/m, surcharge load p = 5.0 kPa, friction angle of the fill $\phi = 45°$ and

Abstract

This paper is based on the publication of Van der Peet and Van Eekelen (2014) and considers the distribution of the vertical load between arching (load part A, in kN/pile or A% in % of the total load) and the residual load parts B + C, in kN/pile. A comparison between numerical results and predictions of three analytical arching models leads to conclusions about the validity and accuracy of these analytical models.

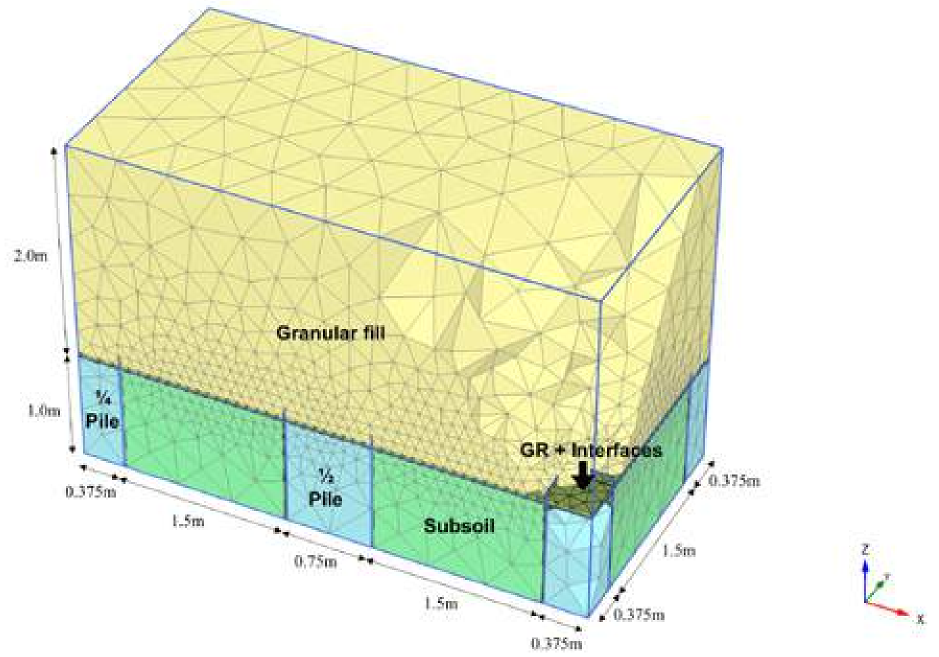

Figure 3 **-** *Basic geometry of the numerical model*

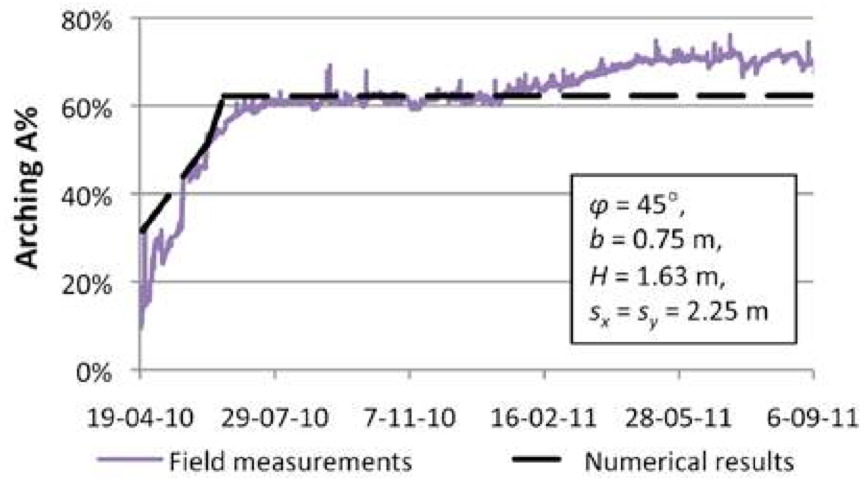

Figure 4 **-** *Comparison of measured load distribution at highway exit Woerden with results of the numerical model*

the embankment height H = 2.0 m. Each of these parameters was individually varied to analyse its influence on the amount of arching. A more elaborate description of the model, including material modelling and phasing is described by Van der Peet et al., 2014.

Scaled model experiments and full-scale field measurements (Van Eekelen et al., 2012a and 2012b) were used to validate the model, as described in Van der Peet (2014). Although the settlements found by the numerical model are small compared to the field measurements, the difference is explicable and acceptable. Moreover, the amount of arching is predicted correctly over the period of construction and use (see Figure 4). The increase in arching in the first months of 2011 is caused by seasonal effects.

RESULTS

The assumption of the analytical models that the stress arches are in limit state is useful for design purposes, since it leads to the lightest construction that will remain stable. In reality however, ultimate limit state (ULS) will not always be reached throughout the embankment. In the numerical model, ULS is only reached when the subsoil does not support the structure. When ULS has been reached, the shape of the arches is round. This is shown by Figure 5, to which the shape of three main arches is added.

The arch that is based on the corners of the piles is strictly semi-circular: it has an equal radius in vertical and horizontal direction. The larger arches, which are based on top of the piles, are higher than they are wide, which makes their shape more elliptical than circular. The smaller arches consistently are wider than high. This is similar to the Zaeske model, in which the arches are wedges that are thicker in the middle than at the sides. Another aspect of these results is however similar to the Concentric Arches model: the smaller arches are based on the GR instead of on the piles.

The numerical model finds a load distribution on the subsurface that is compared with the results of the analytical models in Figure 6. The figure shows clearly that the results of the numerical calculations agree better with the load distribution of the Concentric Arches model than with any of the other models.

The axial stiffness J of the GR was varied between 1000 kN/m and 2500 kN/m. Additionally, all calculations were performed using a bi-axial stiffness (shear stiffness GA equal to zero) and an isotropic stiffness (GA equal to half the axial stiffness). The numerical model finds no influence of the GR stiffness on the amount of arching. This matches all three analytical arching models, since none use the GR stiffness as a parameter.

The surcharge load p, similar to the GR stiffness, does not influence the relative amount of arching A% in any of the three analytical models. However, the numerical results, for variations between 5 kPa and 100 kPa, show that a higher

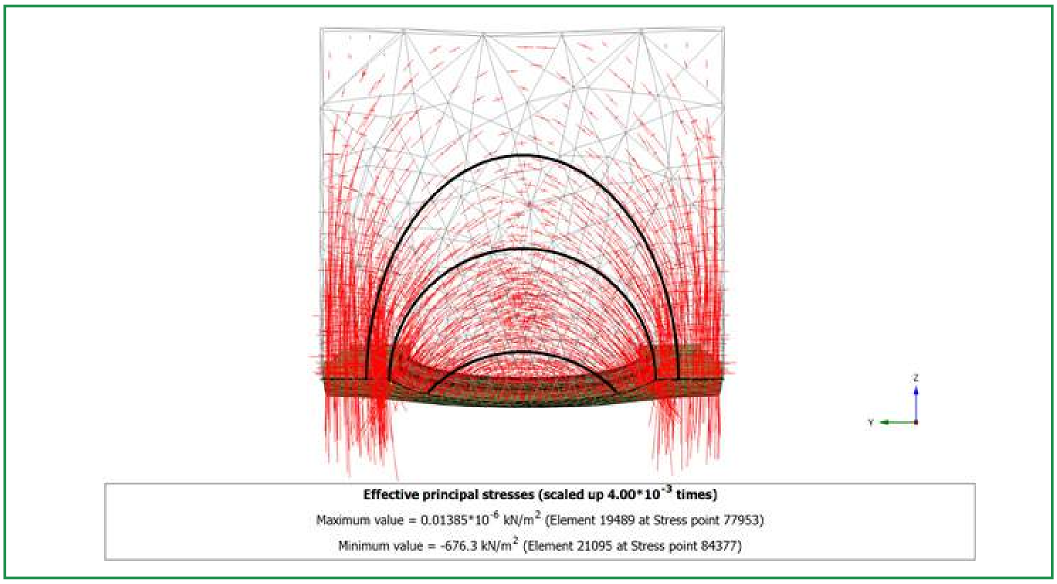

***Figure 5** - Principal stress directions between two neighboring piles in ULS*

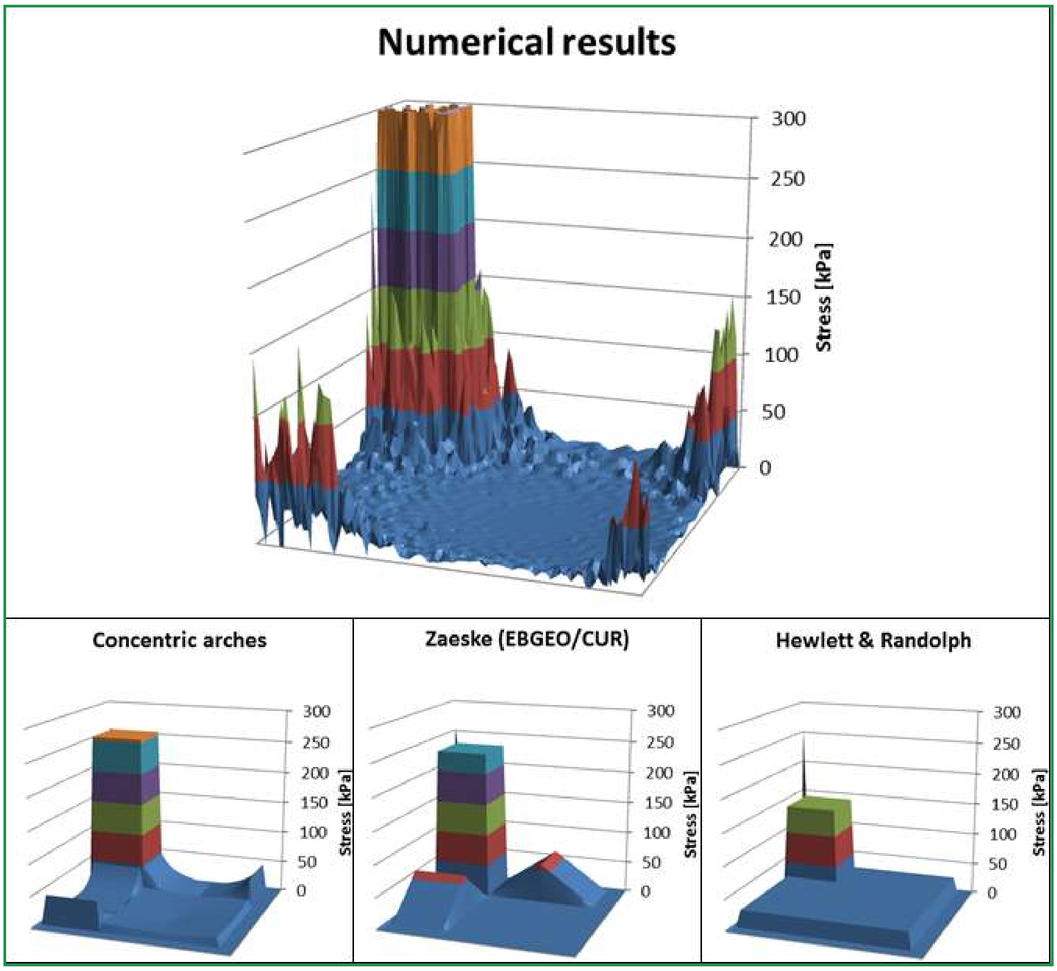

***Figure 6** - Vertical stress on GR resulting from numerical calculations and analytical models*

surcharge load leads to a higher percentage of arching (Figure 7).

The friction angle ϕ of the fill was varied between 30 degrees (low-frictional sand) and 60 degrees (high-frictional crushed rubble material). All three analytical models describe an increase in arching for the higher friction angle, which is supported by the numerical model. The amount of this increase most closely resembles the Concentric Arches model (Figure 8).

The embankment height H was varied between 0.65 and 8 meter. The lower values are smaller than half the open spacing between the piles, which means partial arching will occur. The Hewlett and Randolph model does not include a solution for this situation, while the Concentric Arches model gives an explicit solution for these lower heights. For a more complete picture, this analysis was also done using a friction angle of 35° instead of the basic value of 45°, and for a surcharge load of 30 kPa instead of 5 kPa. The results found in the numerical model are similar to the Concentric Arches model and to a less extent to the Zaeske model, see Figure 9, Figure 10 and Figure 11.

CONCLUSIONS

For all parameter variations, the Concentric Arches model gives satisfactory results. Only the influence of surcharge load is not included in the model. The Zaeske model does not include the influence of surcharge load either. It correctly includes the influence of embankment height, but predicts the influence of the fill's friction angle with less accuracy than the Concentric Arches model. The Hewlett and Randolph model overall leads to far lower amounts of arching than found by numerical analysis. All considered, the Concentric Arches model performs better than the Hewlett and Randolph (1988) model. Compared to the Zaeske (2001) model, the results of the Concentric Arches model are at least similarly accurate and in certain cases better.

ACKNOWLEDGEMENTS

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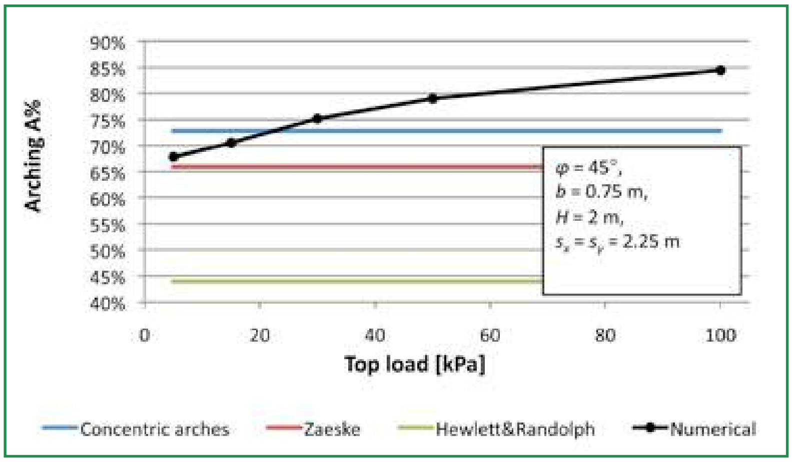

Figure 7 **-** *Relationship between the surcharge load p and arching A% in numerical and analytical calculations*

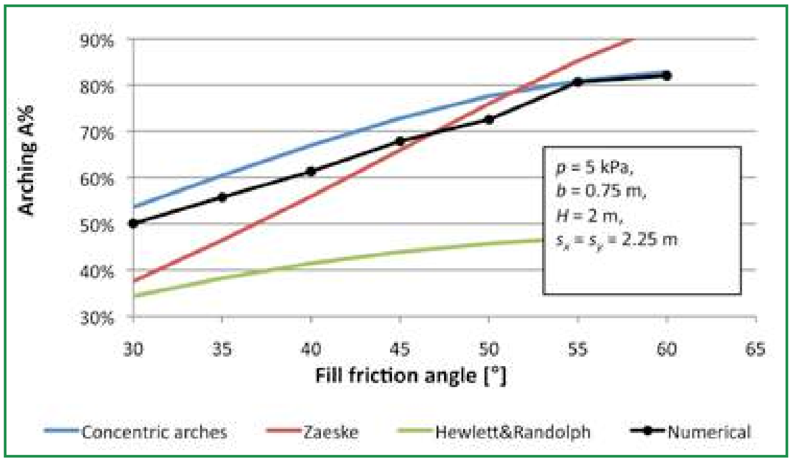

Figure 8 **-** *Relationship between friction angle (φ)and arching A% in numerical and analytical calculations*

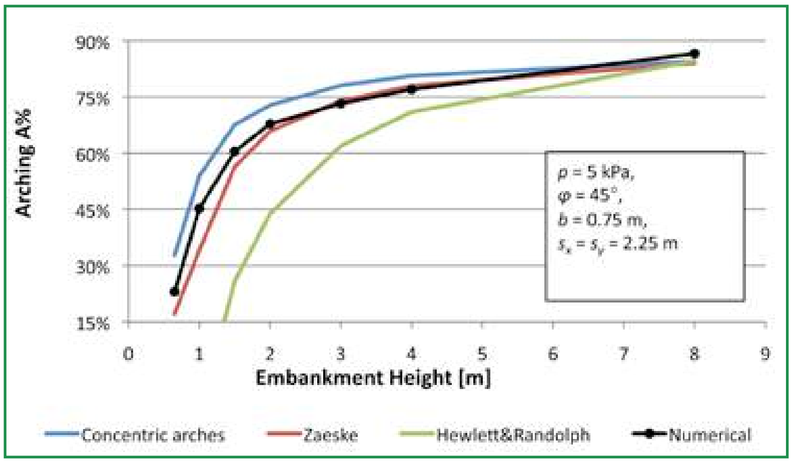

Figure 9 **-** *Relationship between embankment height H and arching A% in numerical and analytical calculations*

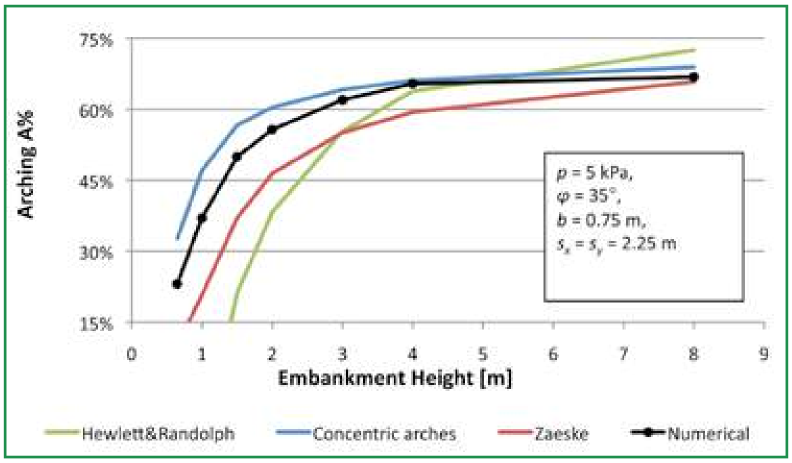

Figure 10 **-** *Relationship between embankment height H and arching A%, for a friction angle ϕ=35°.*

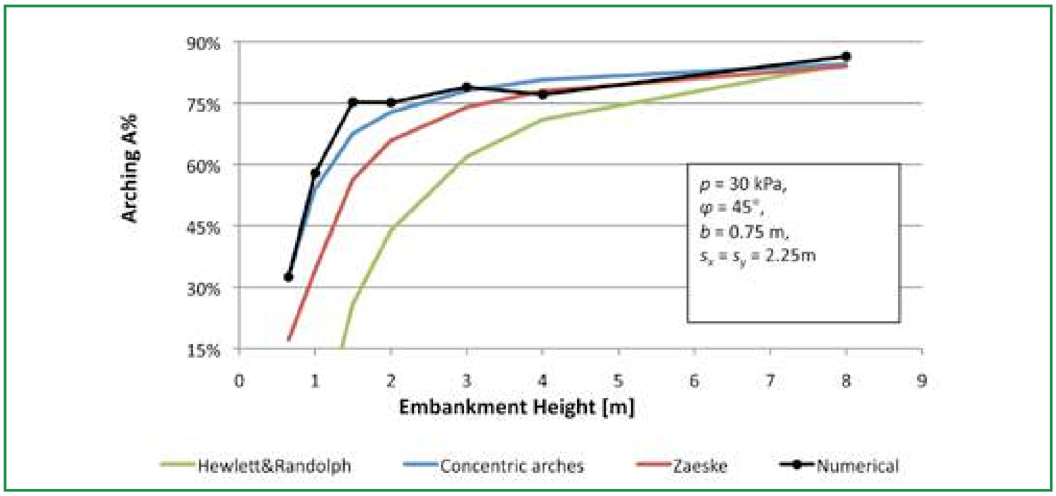

Figure 11 **-** *Relationship between embankment height H and arching A%, for a surcharge load p=30kPa*

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