FINAL REPORT

Hydraulic modelling of pile field groyne bank stabilisation to inform design guidelines

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1 Introduction

1.1 Overview

Alluvium Consulting Australia Pty Ltd (Alluvium) have been engaged to undertake advanced hydrodynamic modelling of pile field groyne bank stabilisation works, to inform design guidelines. The project is funded by the partnership between the Australian Government's Reef Trust and the Great Barrier Reef Foundation.
The aim of this project is to improve the understanding of the hydraulic effectiveness of pile field groynes used for stream bank stabilisation within Great Barrier Reef (GBR) catchments. The outputs of this study will assist in advancing design approaches which may result in a more efficient allocation of funding to stream bank stabilisation and more cost-effective sediment reduction programs within GBR catchments.

This project aims to explore improvements to the design of pile field groynes for stream bank stabilisation using two- dimensional (2-D) hydraulic modelling informed by three-dimensional (3-D) hydrodynamic modelling.

1.2 Project background

Pile field groynes have been extensively used for stream bank stabilisation across the GBR catchments, accounting for over $10 million estimated expenditure in the past five years. These include major projects within the O'Connell River, Mary River and Fitzroy River, (Figure 1).

The aim of pile fields is to reduce near bank velocity and shear stress and promote sediment deposition and vegetation establishment. Timber piles have a finite design life (typically 10-15 years). The design life is typically sufficient for the establishment of deep rooted vegetation along the bank, intended to provide the long-term bank stability (Figure 2). The establishment of vegetation relies on the pile fields creating the hydraulic conditions suitable for vegetation establishment (i.e. reducing the high velocity and high shear stress flow from the toe of an eroding bank).

Current design approaches are outlined in the Technical Guidelines for Waterway Management which were developed by the Victorian Government in 2008. Two design approaches are presented in the guidelines:

- The notional line of attack approach which is neither precise nor determinate and has no basis in stream hydraulics or geomorphic processes. Despite the absence of theoretical basis for the 'notional lines of attack', the approach has proved effective in practice.
- The sheer stress reduction approach which is based on a combination of
    - research undertaken by Dyer (1995) using a straight flume, and
    - results of research into the effective energy gradient on meander bends in stream meanders of varying radius of curvature.

Both of these approaches have strong anecdotal evidence of success, however this may be due to the design approaches being overly conservative (i.e. provision of more piles than required).

In addition to the approaches set out above, 2D hydrodynamic modelling is typically used by practitioners to assess the hydraulic impacts of pile fields. However, 2D models are unable to simulate complex 3D flow patterns associated with pile fields. These flow characteristics are key drivers of scour and riverbank erosion and have a significant impact on pile field performance. These complex flow patterns can be simulated by 3D modelling. However, it is not feasible to use time intensive 3D models at each river restoration site.

However, advancements in 3D hydrodynamic modelling in recent years provide the opportunity to improve our understanding of the complex flow responses surrounding these permeable groyne structures.

This study has used 3D modelling to improve our understanding of the complex flow responses around permeable groynes, and to inform more readily accessible 2D hydraulic models.

Seven river reaches across GBR catchments were modelled as part of this study. The study investigates the hydraulic impact of several pile field design elements and also assesses the impact of geomorphic factors such as river width, flow depth, and bend sinuosity on the hydraulic effectiveness of pile fields. The outcome of this work explores the relationship between pile field configuration and shear stress reduction, to aid more cost-effective design. The results of the investigations can inform the development of guidelines to assist practitioners understanding of the potential impacts of pile fields on river hydraulics and sediment transport processes.

Figure 1. *One of the largest river bank stabilisation project in Australia used pile field groynes on the Fitzroy River.*

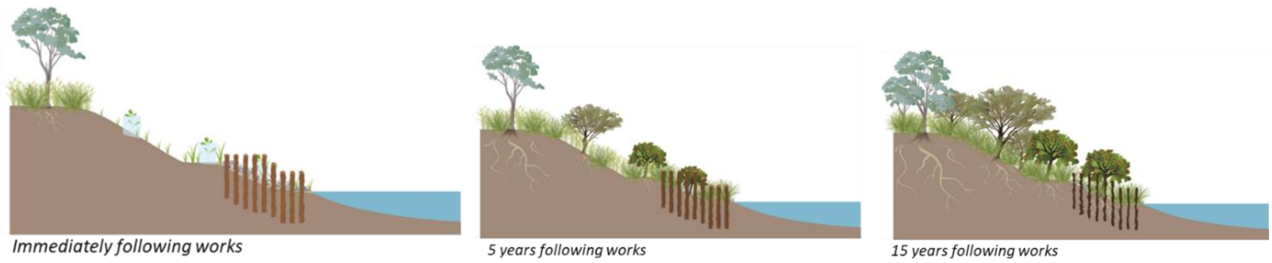

Figure 2. *Changes to vegetation condition over time as a result of the pile field groynes*.

1.3 Report structure

This report has the following structure:

- **Section 1 Introduction -** Provides project background and overview.
- **Section 2 Literature review –** Provides background on the current literature and understanding of the hydraulic and geomorphic effects of permeable groyne structures (such as pile fields).
- **Section 3 Review of existing groyne design guidelines –** Outlines limitations of current design guidelines**.**
- **Section 4 Research methodology –** Outlines the research methodology including modelling approaches, site selection and model setup.

- **Section 5 Results –** Presents the hydraulic modelling results.
- **Section 6 Discussion–** Presents the discussion.
- **Section 7 Design guidelines –** Outlines recommendations and considerations for the design of pile fields groynes.
- **Section 8 Summary and future recommendations –** Outlines future recommendations.

2 Literature review

2.1 Overview of groynes

Groynes (also known as lateral dikes or spur dikes) are placed in sequence along streambanks to disrupt near bank flow lines and reduce bank erosion. There are two key types of groynes:

1. **Impermeable groynes** are solid structures designed to block and deflect flow, and can be constructed from rock, concrete, gravel or sediment.
2. **Permeable groynes** are vertical fence structures which allow water to flow through the structure at a reduced velocity (Przedwojski et al., 1995, Zhang et. al., 2009a). Permeable groynes can be constructed from timber, bamboo, or steel piles, with or without horizontal rails or cables (DSE, 2007).

Both types of groynes are shown in Figure 3.

Figure 3. *Example of impermeable rock groynes in Italy (left) (Sukhodolov, 2017), and permeable pile field groynes in Australia (right).*

Impermeable groynes are commonly used for river training. By modifying flow distribution and pushing the thalweg away from the bank, impermeable groynes can be used to protect the riverbank and improve navigational conditions (i.e. increase water depth) (Przedwojski et al., 1995, Kang et al., 2011). In contrast, permeable groynes are primarily used to mitigate bank erosion, and have less impact on downstream and adjacent river hydraulics compared to impermeable groynes.

By impeding flow, impermeable groynes create a complex 3D separation of flow upstream of the structure and vortex shedding (oscillating circular flow) downstream of the structure. These flow characteristics create localised zones of high shear stress and turbulence resulting in localised scour (Teraguchi, 2011). By allowing water to flow through the structures, permeable groynes reduce flow velocity and turbulence proximal to the structure and can be used to minimise scour around groynes. Permeable groynes also facilitate longitudinal transport and deposition of suspended sediment through the groyne structures. Due to sediment deposition within permeable groyne embayments, suspended concentration decreases in the downstream direction. In contrast, reduction in suspended sediment concentration is not characteristic of impermeable groynes (Gu et. al., 2011). This investigation is focussed on permeable groynes.

Permeable groynes – pile fields

Pile fields, constructed from timber piles, are one example of permeable groyne structures which have been used extensively in riverbank stabilisation projects in Australia. A pile field comprises individual timber piles which are embedded into the riverbank (Figure 4). Pile fields are spaced to optimise shear stress reduction and are oriented with the riverward end pointed downstream directing flow away from the eroding bank.

The primary function of pile fields is to reduce near bank flow velocity and shear stress. Groynes increase flow resistance, decelerate flow, and promote deposition of fine and coarse sediment within embayments (the area between two groynes) (Rutherford, 2007) (Figure 5). Reduction of near bank velocity results in a reduction in shear stress (i.e. hydraulic force) acting on a streambank, and therefore a reduction in fluvial scour (Blackham, 2006).

The following sections provide an overview of:

i. the impacts of permeable groynes of river processes and bank stability, and

ii. key design and environmental factors which impact groyne performance.

Figure 4. *Recently constructed pile fields on the Mary River, Kenilworth.*

Figure 5. *Permeable pile fields along Upper Brisbane River, Harlin, showing reduced flow velocities within embayments.*

2.2 Flow processes around pile fields

The complex flow, turbulence and sediment transport processes which occur through and around pile fields are illustrated in Figure 6. These flow processes are due to the adverse pressure gradient immediately upstream of the pile which induces boundary layer separation and results in a horseshoe vortex at the base of the piles (caused by downward rotation of flow) (Huang, 2014). The subsequent formation of wake vortex (oscillating circular flow) downstream of the piles results in energy loss, reduction in flow velocity and as a result a reduction in shear stress which may lead to sediment deposition within the pile field embayments. The horseshoe vortex also increases bed shear stress in front of and between the piles resulting in localised scour (Li, 2018).

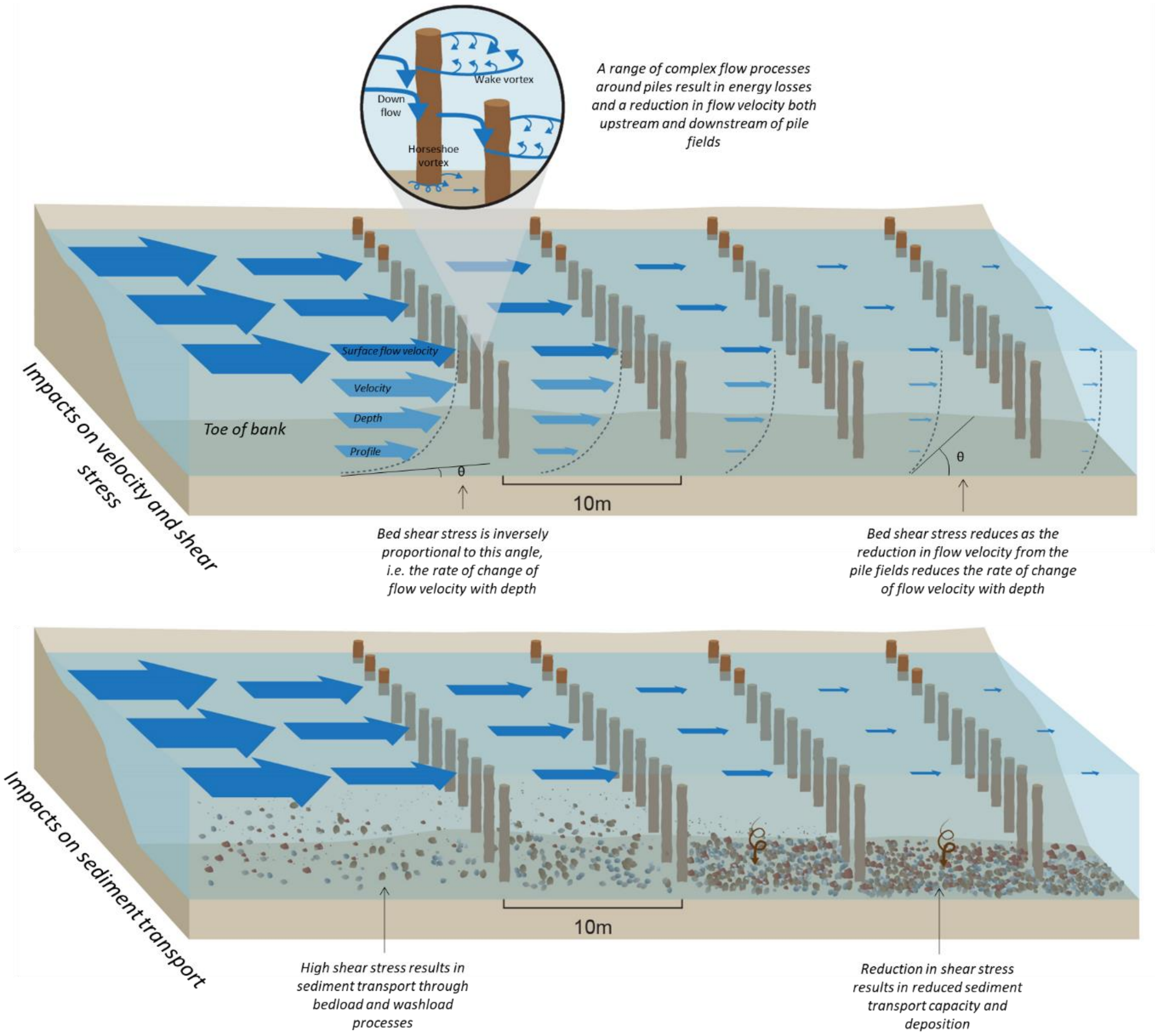

Figure 6. *Flow processes around pile fields. Impacts on velocity and shear stress (top), impacts on sediment transport (bottom).*

2.3 Impacts of groynes on river processes

Permeable groynes impact on the hydrodynamics and morphodynamics of rivers through three key processes:

1. Local scour near groynes,
2. Deposition within embayments, and
3. Modification of velocity distribution across the river channel.

A summary of these processes is provided below.

Local scour near groynes

Pile fields are used to reduce flow energy and consequently fluvial scour in the near bank zone; converting sites of scour to depositional environments (DSE, 2007). However, pile field structures themselves are also subject to scour. Pile fields restrict flow, creating a backwater effect which influences the pressure distribution around the structure. This change in pressure distribution can result in the formation of eddies and consequently local scour near individual piles.

A straight flume study by Teraguchi (2011) found that scour occurs uniformly around permeable piles. Flume studies have also found that scour initiates at the riverward end of permeable groynes, due to flow separation and the formation of eddies (Zhang et. al., 2009a and Dyer et al., 1995). This finding is supported by field observation (SCRC, 1991). The addition of a tail angled 30 to 45 degrees to flow, at the riverward end of pile fields, reduces eddies and scour nearby the structure and redirecting scour formation to the channel centre (Dyer, 1995).

Deposition within embayments

A primary function of permeable groynes is to mitigate bank erosion by reducing near bank flow velocity and increasing fine and coarse sediment deposition (Alauddin, 2011). Reduction of flow velocity within pile field embayments results in deposition and accumulation of sediments and seeds, creating favourable conditions for riparian vegetation establishment (Carling, et. al, 1996) (Figure 7). The establishment of vegetation increases hydraulic roughness within the pile field embayments, assists in the reduction of near bank flow velocity, and stabilises deposited sediments (Rutherford, 2007).

Figure 7. *Example of vegetation establishment within the embayments of pile fields at Black Range Creek in North East Victoria.*

Velocity distribution

Permeable pile fields impact velocity distribution across the river channel (Figure 8). In a flume study Zhang (2009a) found that free surface flow velocities are reduced when flow approaches and passes through permeable groynes. Conversely, flow velocities in the main channel are significantly increased. A gradual reduction in velocity as flow entered succeeding embayments was also reported in a 3D numerical study by Shampa (2020). A higher velocity band generally occurs at the riverward end of the pile fields (Gu, 2011, Zang, 2009b).

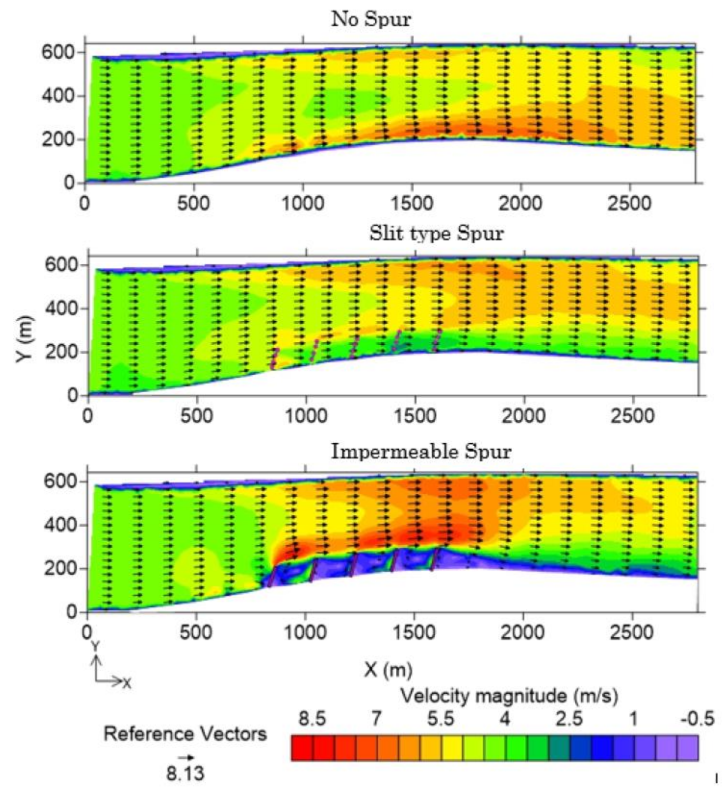

Figure 8. *Distribution of the spatial velocity (m/s). No pile field (top), permable pile field (middle), impermeable pile field (bottom)(from Shampa, 2020)*

The distribution of the transverse velocity (across a pile field) is shown in Figure 9. Pile fields reduce bank-directed transverse velocity. The recirculating cells indicate the presence of a horseshoe vortex and a detached shear layer adjacent to the pile fields. Flow separation is more prominent in the impermeable case compared to the permeable case. The near-bed recirculation is due to the overlapping horseshoe vortex, whereas the upper recirculation is due to the detached shear layer (Shampa, 2020).

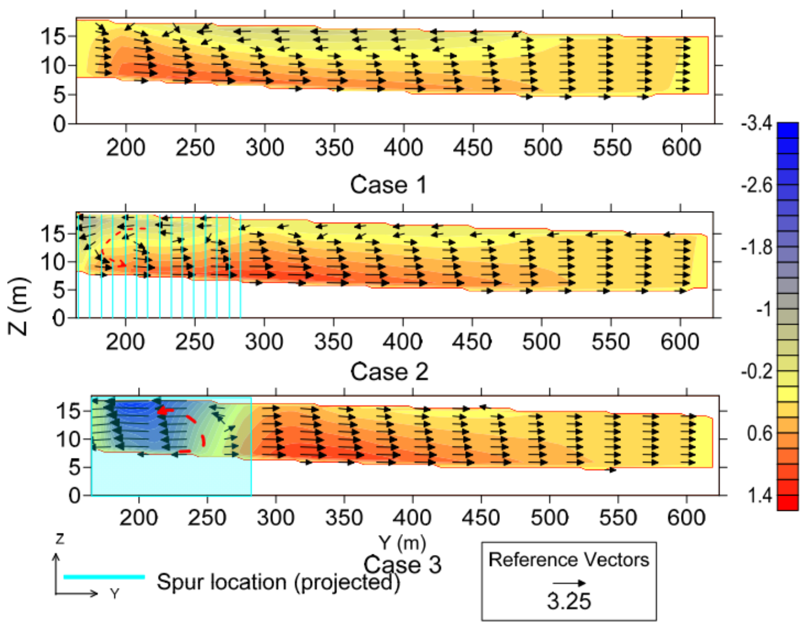

Figure 9. *Distribution of the transverse velocity in m/s across a river cross-section (from Shampa, 2020). No groyne (top), Permeable groyne (middle), impermeable groyne (bottom).*

Groynes also disrupt the erosive helical flow pattern characteristic of meander bends (Abad et al., 2008; Jia et al., 2009). This helical flow pattern is due to the cross-sectional velocity gradient and the outward directed centrifugal force through a meander (Bigham, 2020) (Figure 10).

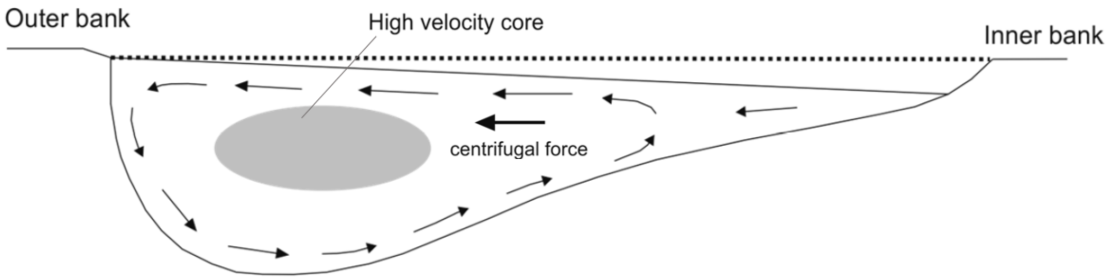

Figure 10. *A schematic of flow characteristics through the apex of a meander bend, with arrows depicting secondary transverse flow direction (from Kasvi, 2017).*

2.4 Impacts of groynes on bank stability

Pile fields mitigate river bank erosion through a range of mechanisms which are discussed below.

Reduces shear stress along toe of bank

Shear stress is the force exerted against the channel and floodplain boundary during flow events and is directly related to variations in the velocity gradient with depth. Once the boundary material critical shear stress is reached the boundary material may begin to scour. Shear stress increases with depth, therefore higher forces are exerted on the channel bed and toe of bank compared to areas higher on the bank profile. Fluvial scour along the toe of bank can lead to a steepening of the bank profile and associated mass failure of the river bank. Pile fields function by reducing flow velocity and applied shear stress along the toe of the bank; mitigating undercutting and steepening of banks due to fluvial scour. This mitigating effect can create hydraulic conditions suitable for vegetation establishment on the bank profile as this area is no longer prone to scour and mass failure process.

Promotes vegetation establishment

By reducing flow velocity pile fields increase sediment deposition along the toe of bank. The reduction in flow velocity and the collection of sediment (and often seeds) in this zone can promote vegetation establishment. The establishment of vegetation along the lower bank can help provide long term stability to the system beyond the design life of the groynes via the following mechanisms:

- The establishment of vegetation increases hydraulic roughness within the pile field embayments, which results in a reduction in near bank flow velocity and therefore a reduction in shear stress acting upon the bank (Rutherford, 2007, and Blackham, 2006).
- Vegetation and root reinforcement can add geotechnical stability by increasing the effective cohesion of the bank material; the level of reinforcement is dependent on root strength and density (Abernethy & Rutherfurd, 1999).
- By providing extensive coverage of bank surface area shrubs and grasses can limit entrainment of bank sediment.

Role of bank reprofiling

Bank reprofiling is often undertaken in conjunction with pile field design and construction. Reprofiling involves modifying a steep riverbank to form a stable slope that is less vulnerable to gravitational mass failure and more suitable for vegetation establishment. Mass failure (i.e. bank slumping) occurs when gravitational forces due to the weight of the bank exceed the cohesive forces within the bank (Osman et. al.,1988). Bank reprofiling reduces risks associated with pile field design and provides more favourable conditions for vegetation establishment. However, bank profiling is associated with a higher design and construction costs.

2.5 Key design factors which impact on pile field performance

Groyne spacing

Groyne spacing impacts shear stress reduction. A straight flume study by Dyer et al. (1995) found that pile fields reduce velocity, downstream of the structure, for a distance up to 40 times the pile height (depending on flow depth). Peng et. al. (1997) also found that a decrease in the distance between groynes decreases the overall bed shear stress between the groynes. In addition, pile fields should be spaced to ensure that flow passing downstream from one pile field is intercepted by the subsequent pile field (Johnson et. al., 1993). This results in a cumulative shear stress reduction in a downstream direction (Figure 11).

Optimisation of pile field spacing should enable target shear stress, for the design flow, to be achieved within the first several pile fields. To achieve target shear stress, the most upstream pile fields are generally closer together, with spacing gradually increasing downstream. As spacing decreases so does associated construction costs. Therefore, optimisation of groynes spacing is key to reducing costs associated with bank stabilisation.

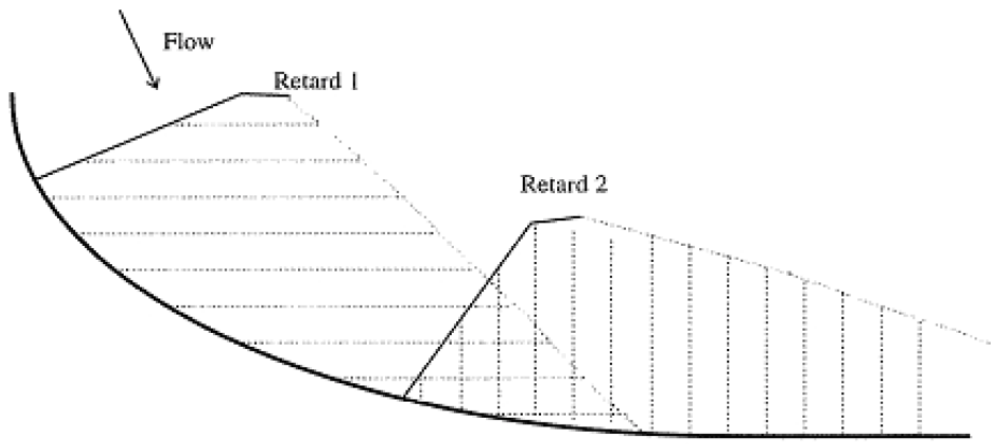

Figure 11. *Cumulative impact of pile fields on flow (from DES, 2007).*

Groyne and pile height

The pile height above the riverbed impacts the downstream distance influenced by the pile field (i.e. shear stress reduction) (DSE, 2007). However, the ratio of pile height to flow depth will impact on the downstream hydraulic impact of the groyne. The *Technical Guidelines for Waterway Management* (DSE, 2007) recommends that the pile height is the lessor of one third of the annual flood stage and annual bankfull stage with additional height applied near bank (guidance based on designer experience rather than hydraulics) .

The elevation (height up the bank) of the most landward pile is dependent on the design flow and acceptable design shear stress along the bank. If the pile fields do not extend far enough up the bank face scour of the upper bank may occur, potentially outflanking the works

Groyne porosity

A straight flume study by Dyer (1995) found that pile field porosity (i.e. ratio of pile cross-sectional area to open area between individual piles) influences velocity profiles in embayments. For examples, a pile field porosity of 50% results in lower relative velocity in downstream embayments than a porosity of 60% (Figure 12). Correspondingly, Kang et. al. (2011) found that a porosity greater than 60% does not reduce velocity in the recirculation zone (downstream of the structure).

Nasrollahi (2008) showed that pile field porosity also has considerable influence on the maximum scour depth. As porosity increases, the scour depth decreases. Similarly, Kang et. al. (2011) demonstrated that a porosity less than 40% creates erosive downward flows. Therefore, design should consider the impact of porosity on both near bank velocity and scour.

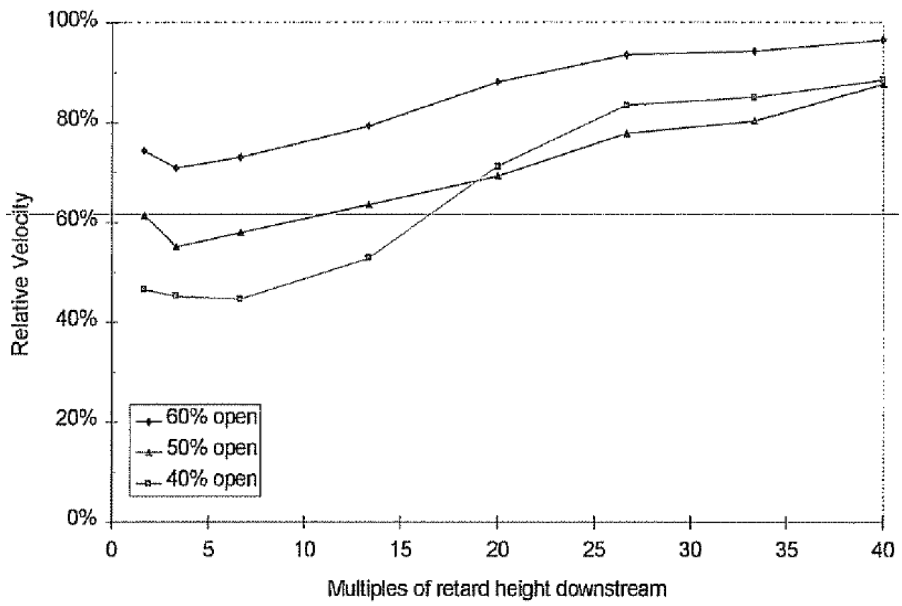

Figure 12. *Velocity profiles in embayments relative to the groyne longitudinal spacing and porosity (% open). (note: "Multiples of retard height downstream" is the longitudinal spacing between groynes measured in a downstream direction) (from Dyer, 1995).*

Groyne angle
The straight flume study by Dyer (1995) found that the angle of the permeable groyne to flow does not have significant impact on the performance of the groyne. However, a slight downstream orientation allows for debris shedding. SCRC (1991) also concluded that hydraulic performance of the pile field is not affected by the groyne angle.

Groyne length
Groyne length is dependent on the desired degree of alignment training. A long pile field which extends past the existing toe of bank significantly alters cross-sectionally hydraulics, pushing the thalweg away from the toe of bank and realigning the river (Rutherfurd, 2007). Short groynes, which generally extend to the existing toe of bank, provide limited alignment training, however, are cheaper to construct and have less impact on river processes.

Embedment depth
Timber piles must be embedded to a depth that will enable the pile to withstand drag, impact and resistance forces within a river (DSE, 2007). Maximum scour depth around piles should inform design embedment depth. Abam (1995) found depth of embedment to be the most important of both design and environmental factors on groyne stability. Increased embedment depth can assist in providing long term groyne stability.

Groyne material
Permeable groynes can be constructed from timber, bamboo, concrete or steel piles. The groyne material must have sufficient structure strength to withstand both hydraulic and debris forces without critical failure (DSE, 2007). In tidal reaches the design life timber structures may be reduced due to attack (and consumption) by marine borers (Ivezich, 2012). Marine grade timber can reduce the risk of decay in these environments.

2.6 Key environmental factors which impact on pile field performance

Flow regime
Water level, flow velocity, and consequently flow discharge, influence pile field performance. In most river systems, discharge varies throughout hydrological seasons. The seasonality of flows and flood frequency can have significant impact of groyne stability.

Pile field performance has been found to be sub-optimal in reaches with prolonged sustained flow periods and low sediment supply, (Rutherfurd et. al., 2007). This is due to prolonged periods of low to moderate flow where the critical shear stress of the boundary sediment within the embayments is slightly exceeded. Over extended periods this can result in significant sediment mobilisation within pile field embayments.

In addition, high flow velocities increase the dynamic force on pile fields, this force acts to overturn individual piles. Stability is achieved by ensuring lateral earth pressure acting on the pile exceeds the dynamic flow force (Abam, 1995). The lateral earth pressure relates to the embedment depth.

Sediment supply regime

Sediment supply and transport within river systems is key to pile field performance. High bed and suspended sediment loads provide the requisite supply of sediment for deposition within permeable groyne embayments (Alauddin, 2011). In turn, aggregation of sediments creates favourable conditions for the establishment of vegetation (Carling, et. al, 1996). In conditions of low sediment supply there will be limited deposition within embayments, and vegetation may struggle to establish. Timber piles have a finite design life (typically 10-15 years). Therefore, without the establishment of vegetation along the bank the works will be unable to provide long-term bank stability.

Bioregion

Bioregion characteristics including climate, soil type and vegetation influences pile field performance. Climatic conditions and soil type influence the ability of vegetation to establish, and subsequently support the structural works by increasing critical shear strength of the channel boundary and structural stability of the bank. The establishment of vegetation is a critical component of pile field performance in the long term. Without vegetation establishment the pile fields are unlikely to perform their design intent of reducing long term erosion. As a result, these environmental factors can impact on the long-term success of the works regardless of the hydraulic and geomorphic performance of the works.

Revegetation effort

Vegetation forms a critical component of stream bank stabilisation works. Without the appropriate investment in revegetation design, monitoring and maintenance the pile fields are unlikely to perform their design intent of reducing long term erosion.

3 Existing groyne design approaches and guidelines

There are three common approaches to pile field design used within Australia:

1. the notional line of attack approach,
2. the shear stress approach, and
3. 2D modelling

The notional line of attack approach is detailed in the *Guidelines for Stabilising Waterways* (SCRC, 1991). This approach was developed based on the authors knowledge and experience (DSE, 2007). The shear stress approach was developed based on both research undertaken by Dyer (1995) using a straight flume with alternative pile field configurations, and research into the effective energy gradient on stream meander bends of varying radius of curvature. These two approaches are detailed in the *Technical Guidelines for Waterway Management* (DSE, 2007) and are summarised below.

3.1 Notional line of attack approach

This approach is based on disrupting the critical lines of attack around a bend or through a reach. The approach comprises three key steps which are illustrated in Figure 13 (SCRC, 1991):

1. Identify the critical lines of attach (i.e. using aerial imagery).
2. Starting from the upstream extent use the critical lines of attack to inform the angle of each pile field. The pile fields should be angled 5 to 15 degrees downstream to the perpendicular of the line of attack. The hydraulic performance of the pile fields is not affected by the groyne angle. However, the pile fields should be angled downstream to enable debris shedding.
3. The pile fields should also be oriented to ensure the landward end of each pile field is upstream of the intersection of the critical line of attack from the preceding (upstream) pile field and the streambank.

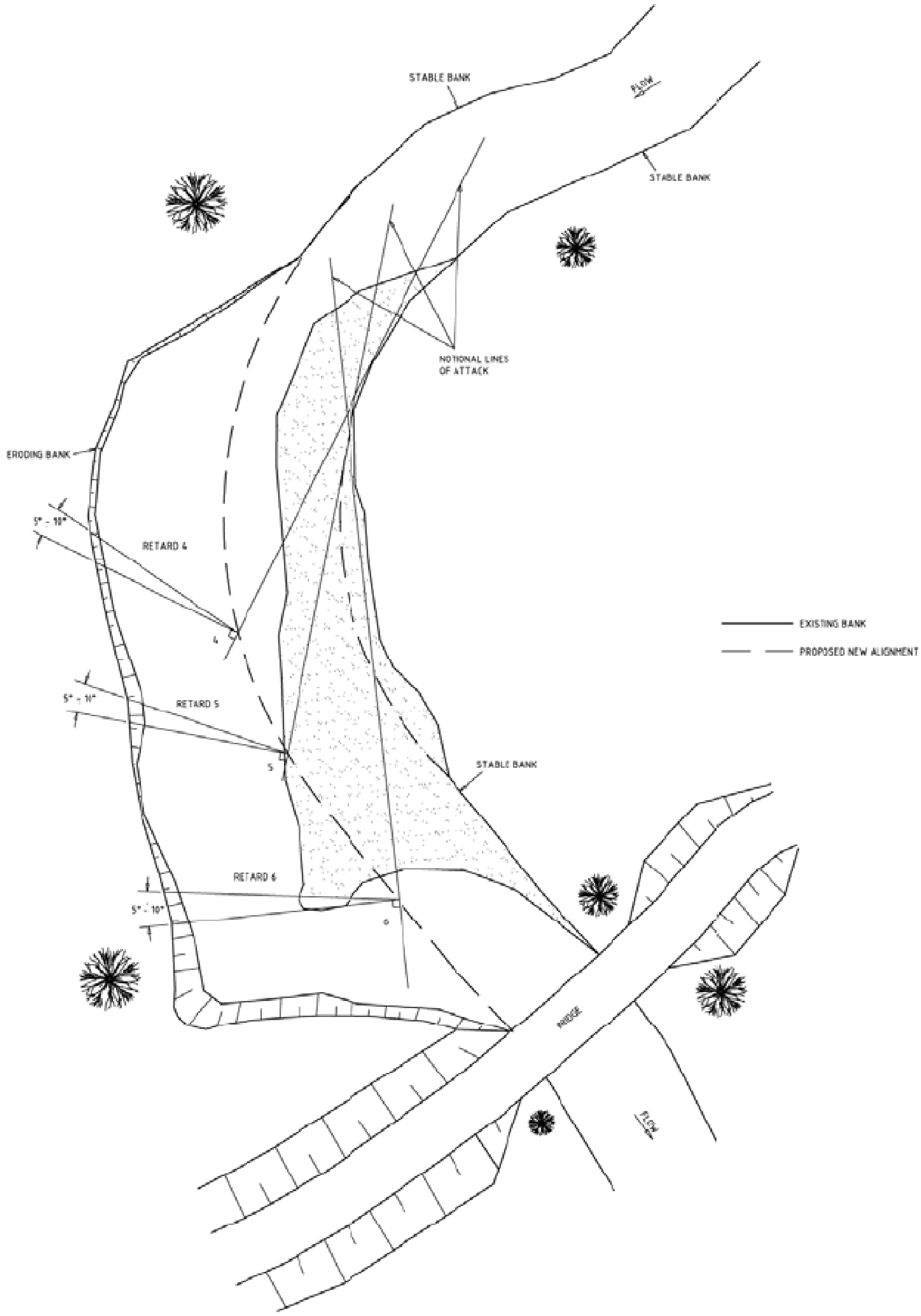

Figure 13. *Schematic indicating pile field placement (spacing and orientation) based on notional lines of attack.*

3.2 Shear stress approach

The shear stress approach can be used in conjunction with the notional lines of attack approach to develop a conservative design. For example, the notional line of attack approach can be used to inform the orientation (angle) of the pile fields, and the shear stress approach to inform the spacing of the pile field groynes. Pile field spacing, and length, accounts for the river width and depth, the radius of curvature of the river bend at the study site, direction and velocity of design flow, and the desired velocity in the embayments.

The shear stress approach to the design of pile fields comprises 8 key steps (DES, 2007):

1. Identify acceptable design risk for the site based on the vegetation establishment period, and likelihood of an event occurring within this period which would potentially mobilise sediment and cause the proposed works to fail.
2. Identify the design flow associated with the design criteria in Step 1.
3. Identify the target shear stress for the design flow based on the critical shear stress of the bed and bank sediments.
4. Estimated the design shear stress for the design event (i.e. using a one-dimensional (1-D) or 2D model).
5. Adjust the effective shear stress to account for the radius of curvature within the study area.
6. Determine target shear stress reduction based on effective shear stress and target design shear stress
7. Determine the downstream extent of shear stress reduction achieved by the first (most upstream) pile field based on pile porosity, height and distance from the bank. Use shear stress reduction to inform the distance to the next downstream pile field.
8. Analysis of the spacing of subsequent downstream pile fields should be informed by the shear stress reduction of the preceding (upstream) pile field.

3.3 Two dimensional hydraulic modelling

2D modelling is often used for pile field design, however no design guidelines were identified that provide an approach to design based on 2D hydraulic modelling. The concept used in design has been to reduce the shear stress, for design flow events, within the embayments, to below the critical shear stress for the mobilisation of bed and bank material. A number of issues arise with this approach including, the ability to scale the model cell size to reflect the pile size, the selection and analysis of suitable design flow events and the validity of 2D modelled bed shear stress.

Cell sizing: Recent advancement in 2D modelling have now allowed variable grid sizes which allow small cell sizes (i.e. 0.1 m) surrounding piles to more accurately simulate flow dynamics surrounding the groynes. Prior to this advancement it was difficult to accurately model piles due to the large number of cells required which increased computational requirements.

Design flow events: The issue of design flow events has been resolved to a large extent through the development of cumulative failure analysis (Section 7.3).

2D model validity: There is also some uncertainty surrounding the calculation of shear stress in 2D models and how this compares to published shear stress thresholds which were developed from one-dimensional, reach averaged, model outputs. Hydraulic parameters which relate to energy losses (such as shear stress) will typically be higher in 2D model outputs due to differences in the treatment of these energy losses (compared to one-dimensional analysis). This issue requires further research to help determine shear stress thresholds for 2D model outputs.

3.4 Summary of design approaches

Both the notional line of attack and shear stress approach have strong anecdotal evidence of success; however, this success may be due to the design approaches being overly conservative (i.e. more piles are used than are required).

There are several challenges associated with modelling pile fields in one-dimensional and two-dimensional models. 1-D models can simulate the magnitude of hydraulic parameters, such as velocity and shear stress, at delineated channel cross-sections. However, 1-D models are unable to model:

i. Variations in hydraulic parameters across the section and in the areas between cross-sections, or

ii. The hydraulic effect of changes in cross-sectional geometry and meander bends.

Consequently, 1-D models are not able to directly simulate complex flow patterns associated with pile fields such as zones of accelerated flow between piles, and increased shear stress at outer meander bends.

2D models can simulate both the flow direction and velocity at hydraulics structures. Therefore, 2D models can more accurately simulate hydraulic behaviour of flows around pile fields. Accuracy of 2D modelling of pile fields is largely dependent on the selected model cell size and time step, and the shape and size of the elevation shape used to simulate the pile fields. However, like 1-D models, 2D models are unable to simulate complex 3D flow (depth) patterns associated with pile fields. Such as flow separation and associated scour around pile fields, and the erosive helical flow pattern characteristic of meander bends. These 3D flow characteristics are key drivers of scour and riverbank erosion and have considerable impact on pile field performance.

Advancements in 3D hydrodynamic modelling in recent years provide the opportunity to improve our understanding of the complex flow responses surrounding these impermeable groyne structures.

4 Research methodology

This section outlines the research methodology undertaken for this project including: the hydraulic modelling approaches, pile field site selection, the development of pile field configurations for simulation, and the modelling setup.

4.1 Hydraulic modelling approaches

A range of methods for modelling pile fields were considered as part of this project. An overview of each of the modelling approaches and their strengths and weaknesses in relation to modelling pile fields is discussed below.

CFD model
Computational fluid dynamics (CFD) models attempt to simulate the interaction of fluids around surfaces by solving the full Navier-Stokes equations. CFD methods can broadly be separated into two categories: Eulerian and Lagrangian. In the Eulerian approach the computational domain is discretised by a finite set of points called a grid (or mesh) and the approximate solution is computed at these grid points. In the Lagrangian approach the computational region is discretised by a set of particles which move at the local flow velocity and approximate solutions are computed at the position of each particle at each discrete time. CFD models are often used in the design process for aerodynamic and hydrodynamic processes.

CFD models can be used with grid scales in the order of $O(10^{-2} - 10^{1})$m. CFD models are able to explicitly model all the physical processes involved in flow through a pile field. The biggest disadvantage of CFD models is the huge computational requirements with super computers often being needed.

In this project we initially attempted to solve the Mary's Rivers Carter pile field site using the CFD Model Phoenics (https://www.cham.co.uk/phoenics.php), however due to the extent of the domains and the required memory needed meant that we were unable to get either grid to run with the available computer resources. With sufficient computer power it may be possible to use a CFD model to fully simulate a pile field or potentially a sub-section of a pile field.

3D modelling
3D environmental scale models solve the unsteady Reynolds-averaged Navier-Stokes equations. The major difference between 3D environmental scale models and Computational Fluid Dynamics models (discussed above) is that environmental scale models do not explicitly model the turbulent processes but instead require a sub-grid scale turbulence closure scheme. In this project the 3D model AEM3D has been used. 3D models are used when vertical stratification of the water column becomes important.

The main advantage of 3D models for sediment transport studies is their ability to simulate the vertical structure in velocity and in particular to capture the bottom boundary profile. 3D models are generally used with horizontal grid scales in the order of $O(10^{1} - 10^{3})$m and are not usually setup with grids capable of accurately resolving pile fields. The biggest disadvantage of 3D models over 2D models is the computational expense.

2D modelling
2D models work by solving the two-dimensional depth-averaged Navier-Stokes equations for incompressible fluids. 2D models are most appropriate for situations where the vertical distribution of velocity and scalars (temperature, salinity, nutrients, etc) are insignificant. 2D models are often used in flood modelling studies where it is assumed that the large horizontal velocities act to create turbulence and mix the water column vertically.

The main advantage of 2D models over 3D models is the reduced computational time. Solving the depth integrated momentum and transport equations rather than the full 3D equations is much simpler and allows greater horizontal resolution to be used. A further advantage of 2D models is because there are no vertical layers in the grid structure the bathymetry of the water body can be fully resolved.

The biggest disadvantage of 2D models when simulating sediment transport is that the bottom boundary velocity profile is not resolved. By definition, in a 2D model, the velocity is assumed to be constant over the full depth and the bottom drag force is therefore applied uniformly over the depth. 2D models can also have issues when the river channel is density stratified. Density stratification can occur either due to salinity or temperature effects. When density stratification is important the 2D assumption becomes invalid and the integrated velocity is not representative of the bottom velocity, this in turn effects the calculated shear stress and sediment transport. 2D models also are unable to accurately represent horizontal drag forces at sharp boundaries in the bathymetry or at the boundary with a pile or groyne.

2D and 3D hydraulic modelling have been used for this investigation. The hydraulic modelling approaches adopted for this study were AEM3D and HECRAS2D. An overview of these models is provided in Section 4.4. An analysis of the 3D and 2D hydraulic modelling results is provided in Section 5 and 6.

4.2 Site selection

Overview

A review of pile field sites across Great Barrier Reef catchments and eastern Australia more broadly has been undertaken. A database of these sites was created which outlines the key site parameters including channel width, radius of curvature, bank height, bioregion and sediment regime. All of these sites are existing pile field sites where there is some data to enable additional analysis. From this database, a sample of sites have been selected for the modelling and analysis component of the project.

Six sites have been identified within GBR catchments for this study. Selected sites have unique hydro-geomorphic characteristics intended to exemplify the impact of a range parameters on the hydraulic effectiveness of pile field groynes. One additional site was identified on tight meander of the Yarra River (Victoria); this unique hydro-geomorphic characteristic was not otherwise represented by sites within GBR catchments. The study studies are shown in Figure 14 and summarised in Table 1. Each of the seven pile field study sites are discussed below.

The pile field bank stabilisation specifications for each site are provided in Attachment A.

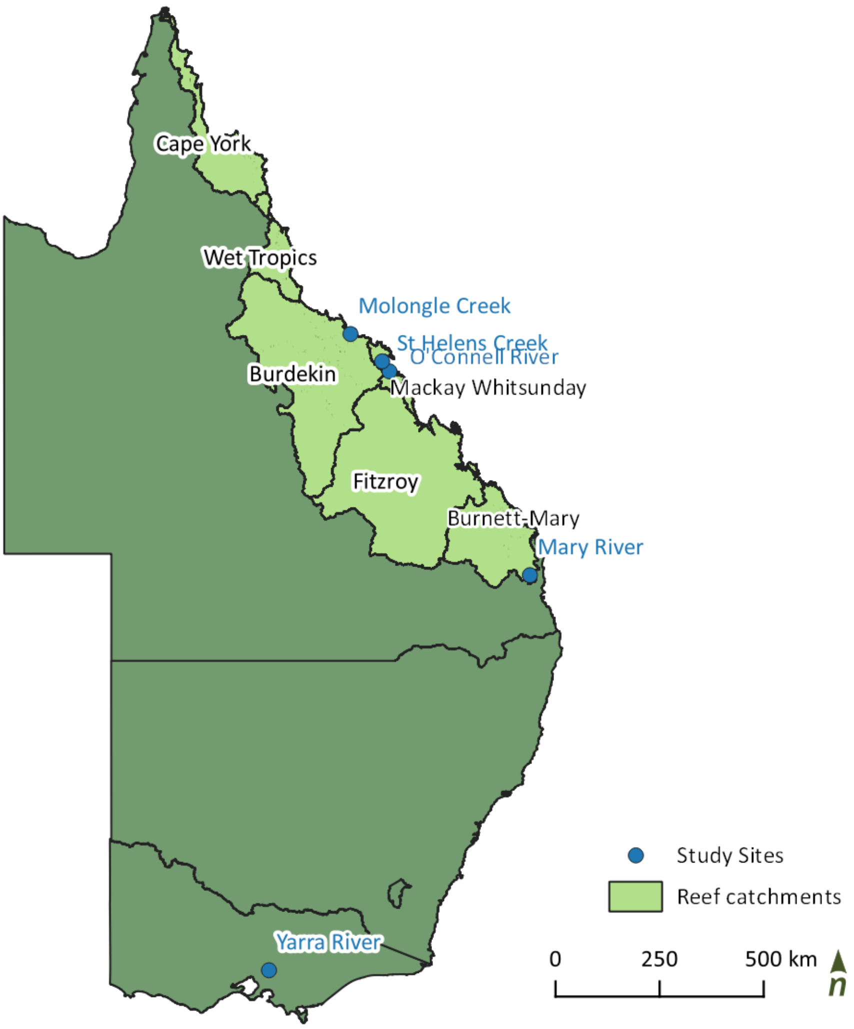

Figure 14. *The location of the seven study sites.*

Table 1. Summary of the study sites.

Study Site	Reef NRM region	Site description
Mary River – Carters	Burnett-Mary	Wide channel, with a medium-sinuosity platform
Mary River – Kenilworth Park	Burnett-Mary	Medium width channel, with a low-sinuosity platform
O'Connell River – Site 1	Mackay Whitsunday	Medium-width channel, and high-sinuosity platform
O'Connell River – Site 2	Mackay Whitsunday	Narrow-width channel, and a low-sinuosity platform
St Helen Creek	Mackay Whitsunday	Medium-width channel, and medium-sinuosity platform
Molongle Creek	Burdekin	Medium-width channel, and medium-sinuosity platform
Yarra River	-	Narrow channel, and high-sinuosity platform

Mary River – Kenilworth Park

The Mary River Kenilworth Park site is located on an outside meander opposite the Obi Obi Creek confluence (approximately 1 km upstream of the Kenilworth Bridge, and 430 m downstream of Carters Site). Prior to reprofiling, the site consisted of a near-vertical bank, approximately 8 m in height (Figure 15).

Figure 15. *The near-vertical bank adjacent to Kenilworth Park Kenilworth (looking upstream).*

The Mary River, adjacent to Kenilworth Park, has a channel width and radius of curvature of 120 m and 504 m respectively. This section of the Mary River is classified as having a medium-width channel, with a low-sinuosity platform. Key hydro-dynamic parameters characteristic of this reach are summarised in Table 2.

Table 2. Key hydro-dynamic parameters – Mary River Kenilworth Park study site.

Site	Reef Catchment	Construction date	Average bank height (m)	Channel width (m)	Width class	Radius curvature (RC)	Ratio RC/ channel width	Sinuosity class
Mary River – Kenilworth Park	Burnett-Mary	2014	8	120	Medium	504	4.2	Low

Mary River – Carters

The Mary River Carters site is located approximately 1.7 km upstream from the Eumundi-Kenilworth Road crossing. Prior to reprofiling, the site consisted of a near-vertical bank, approximately 9 m in height (Figure 16). At the upstream extent of the site, the bank transitioned to a steep bank, approximately 8 m in height. The site abuts a floodplain that sits approximately 3 – 5 m above the inset floodplain immediately upstream of the site.

Figure 16. *The near- vertical bank in the downstream section of the Mary River Carters site (looking upstream) (2017).*

The Mary River, adjacent to Carters site, has a channel width and radius of curvature of 200 m and 320 m respectively. This section of the Mary River is classified as having a wide channel with a medium-sinuosity platform. Key hydro-dynamic parameters characteristic of this reach are summarised in Table 3.

Table 3. Key hydro-dynamic parameters – Mary River Carters study site.

Site	Reef Catchment	Construction date	Average bank height (m)	Channel width (m)	Width class	Radius curvature (RC)	Ratio RC/ channel width	Sinuosity class
Mary River-Carters	Burnett-Mary	2019	9	200	Wide	360	1.8	Medium

O'Connell River Site 1

The O'Connell River Site 1 is located on an outside meander approximately 2.5 km downstream of the Caper's Crossing. The site abuts an inset floodplain which is not used for agricultural purposes. Prior to reprofiling the bank was currently near vertical, with coarse sediment deposition along the toe (Figure 17).

Figure 17. *Looking across-stream at O'Connell River Site 1. The site is characterised by a steep sandy upper bank and gravel deposits at the toe of the bank.*

Table 4. Key hydro-dynamic parameters – O'Connell River study site 1.

Site	Reef Catchment	Construction date	Average bank height (m)	Channel width (m)	Width class	Radius curvature (RC)	Ratio RC/ channel width	Sinuosity class
O'Connell River 1 (A2)	Mackay Whitsunday	2015	7	145	Medium	220	1.5	High

O'Connell River Site 2

The O'Connell River Site 2 is located approximately 2.6 km downstream of the Caper's Crossing (150 m downstream of O'Connell River Site 2). Prior to reprofiling the site consisted of a near-vertical bank, up to 8 m in height, with limited vegetation along the bank face (Figure 18).

Figure 18. *Looking downstream towards O'Connell River Site 2 – the bank is near vertical with predominately grass species growing within the riparian zone.*

The O'Connell River, adjacent to works, has a channel width and radius of curvature of 145 m and 220 m respectively. This section of the O'Connell River is classified as having a medium-width channel, and high-sinuosity platform. Key hydro-dynamic parameters characteristic of this reach are summarised in Table 4.

Table 5. Key hydro-dynamic parameters – O'Connell River study site 2.

Site	Reef Catchment	Construction date	Average bank height (m)	Channel width (m)	Width class	Radius curvature (RC)	Ratio RC/ channel width	Sinuosity class
O'Connell River 2 (A1)	Mackay Whitsunday	2018	8	104	Narrow	-	-	Straight

Molongle Creek

The Molongle Creek site is located downstream of the Bruce Highway, approximately 1.8 km upstream of the creek mouth. The eroded section of bank is approximately 300 m in length and 3.5m in height at the bend and up to 6 m in height upstream of the bend (Figure 19).

Figure 19. *Molongle Creek bank erosion: looking upstream (left), looking downstream (right).*

Molongle Creek, adjacent to works, has a channel width and radius of curvature of 73 m and 196 m respectively. This section of the Molongle Creek is classified as having a medium-width channel, and medium-sinuosity platform. Key hydro-dynamic parameters characteristic of this reach are summarised in Table 6.

Table 6. Key hydro-dynamic parameters – Molongle Creek study site.

Site	Reef Catchment	Construction date	Average bank height (m)	Channel width (m)	Width class	Radius curvature (RC)	Ratio RC/ channel width	Sinuosity class
Molongle Creek	Burdekin	Design stage	3	73	Medium	196	2.7	Medium

St Helens Creek

The St Helens Creek Site is located in lower St Helens Creek, approximately 2 km downstream of the Bruce Highway (Figure 20). Lower St Helens Creek is slightly confined by terraces with extensive zones of inset floodplains.

Figure 20. *St Helens Creek Site (looking upstream) prior to bank stabilisation.*

St Helens Creek, adjacent to works, has a channel width and radius of curvature of 118 m and 299 m respectively. This section of the St Helens Creek is classified as having a medium-width channel, and medium-sinuosity platform. Key hydro-dynamic parameters characteristic of this reach are summarised in Table 7.

Table 7. Key hydro-dynamic parameters – St Helens Creek study site.

Site	Reef Catchment	Construction date	Average bank height (m)	Channel width (m)	Width class	Radius curvature (RC)	Ratio RC/ channel width	Sinuosit y class
St Helens 1 Site 6	Mackay Whitsunday	2018	5	118	Medium	299	2.5	Medium

Yarra River

The Yarra River Site is located on an outside meander bend within the Yering reach of the Yarra River, east of Yarra Glen. Prior to works the bank was poorly vegetated, and consequently provided little resistance to the stream powers exerted upon them in high flow events. This led to undercutting at the toe of the bank and bank slumping (Figure 21).

Figure 21: *Looking upstream at the outside bank of the first meander bend (left), looking downstream at the near vertical and undercut banks (right).*

The Yarra River, adjacent to works, has a channel width and radius of curvature of 25 m and 40 m respectively. This section of the Yarra River is classified as having a narrow channel, and high-sinuosity platform. Key hydro-dynamic parameters characteristic of this reach are summarised in Table 8.

Table 8. Key hydro-dynamic parameters – Yarra River study site.

Site	Reef Catchment	Construction date	Average bank height (m)	Channel width (m)	Width class	Radius curvature (RC)	Ratio RC/ channel width	Sinuosity class
Yarra River	-	2008	-	25	Narrow	40	1.6	High

4.3 Development of pile field configurations

The existing pile fields designs for each site were designed using the shear stress approach to inform pile field spacing, and the notional line of attack approach to inform pile field angle relative to flow. The pile field bank stabilisation works for each site are shown in Figure 22 and Figure 28.

Alternative pile field configurations were developed for each site to test elements of pile field design such as pile field spacing, and height. The alternate designs were informed by the existing pile field design at each site. Six alternative pile field design configurations were developed for each of the sites, and consisted of variation in one of six key design parameters, including:

- Spacing of individual piles (i.e. groyne porosity)
- The number of groynes and spacing between groynes (i.e. distance to downstream pile)
- The exposed height of piles (above the riverbed)
- Groyne length past toe of bank (into channel)
- Groyne angle relative to upstream flow
- Stagger piles (i.e. each pile field comprising two stagger rows)

At each site, the existing design, three design parameters, and six alternative configurations (i.e. two configurations for each design parameter) were assessed. The alternative pile field configurations are intended to exemplify the impact of key design parameters on the hydraulic effectiveness of pile field groynes (i.e. the impact of increase of decreasing pile porosity relative to the existing design). The six key design parameters assessed are illustrated in Figure 29, and the design assessed are summarised in Table 9. The pile field design configurations for the seven sites are summarised in Table 10 to Table 16.

Figure 22. *Mary River Kenilworth pile field bank stabilisation. Aerial view (left), oblique view (right).*

Figure 23. *Mary River Carters pile field bank stabilisation. Aerial view (left), oblique view (right).*

Figure 24. *O'Connell River Site 1 pile field bank stabilisation. Aerial view (left), looking upstream (middle), looking downstream (right).*

Figure 25. *O'Connell River Site 2 pile field bank stabilisation. Aerial view (left), looking upstream (right).*

Figure 26. *St Helens pile field bank stabilisation. Aerial view (left), Looking downstream (right).*

Figure 27. *Molongle River pile field bank stabilisation. Aerial view (left), looking downstream (right).*

Figure 28. *Yarra River pile field bank stabilisation. Aerial view (left), looking downstream (right).*

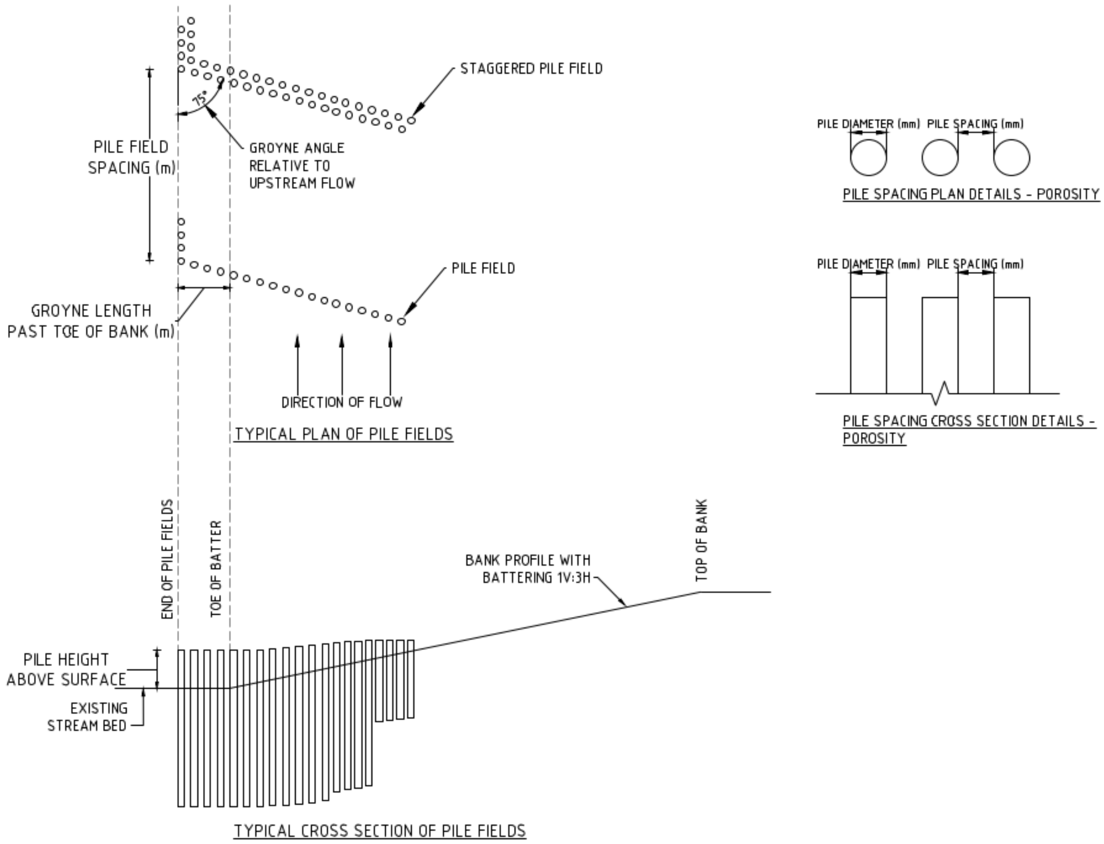

Figure 29. Overview of key pile field design parameters.

Table 9. Summary of pile field design variations.

Pile field design parameter	Variation
Pile field spacing	-26% to +29% existing design (based on shear stress reduction approach)
Porosity (% open)	33, 50, 60, 70%
Pile height above riverbed	1, 1.5, 2, 2.5 m
Groyne angle to upstream flow	45, 65, 90 ^{O}
Groyne length past toe of bank	2 – 12 m

Table 10 Pile field configurations – Mary River Kenilworth (green indicates variation).

Pile field design parameter	Base case	Existing case	Design 1	Design 2	Design 3	Design 4	Design 5	Design 6
Pile field spacing (m)	-	8, 8, 33, 19, 24, 32, 31, 31.	8, 8, 8, 20, 20, 20, 20, 20, 20, 20, 20. (-26%)	15, 30, 35, 35, 35, 35. (+28%)	as per existing	as per existing	as per existing	as per existing
Number of pile fields	-	9	12	7	9	9	9	9
Porosity (% open)	-	50%	50%	50%	50%	50%	50%	50%
Pile diameter (mm)	-	300	300	300	300	300	300	300
Groyne angle	-	65° to flow	65° to flow	65° to flow	90° to flow	45° to flow	65° to flow	65° to flow
Staggered pile fields	-	-	-	-	-	-	Staggered with existing design pile field spacing	Staggered with increased pile field spacing

Table 11. Pile field configurations – Mary River Carters (green indicates variation).

Pile field design parameter	Base case	Existing case	Design 1	Design 2	Design 3	Design 4	Design 5	Design 6
Pile field spacing (m)	-	9, 11, 20, 24, 27, 24, 25, 24.	7.5, 7.5, 7.5, 20, 20, 20, 20, 20, 20, 20. (-19%)	10, 10, 30, 30, 30, 30, 30. (+17%)	as per existing	as per existing	as per existing	as per existing
Number of pile fields	-	9	11	8	9	9	9	9
Porosity (% open)	-	50%	50%	50%	33%	70%	50%	50%
Pile diameter (mm)	-	300	300	300	300	300	300	300
Pile height above riverbed (m)	-	1.5	1.5	1.5	1.5	1.5	1	2

Table 12. Pile field configuration – O'Connell River Site 1 (green indicates variation).

Pile field design parameter	Base case	Existing case	Design 1	Design 2	Design 3	Design 4	Design 5	Design 6
Pile field spacing (m)	-	10, 11, 16, 21, 19, 21.	8, 7.5, 8.5, 11, 17, 17, 16, 16. (-20%)	12, 13, 24, 25, 26. (+19%)	as per existing	as per existing	as per existing	as per existing
Number of pile fields	-	7	9	6	9	9	9	9
Pile height above riverbed (m)	-	1.5	1.5	1.5	2	2.5	1.5	1.5
Porosity (% open)	-	50%	50%	50%	50%	50%	50%	50%
Pile diameter (mm)	-	300	300	300	300	300	300	300
Groyne length past toe	-	4	4	4	4	4	9	12

Table 13. Pile field configuration – O'Connell River Site 2 (green indicates variation).

Pile field design elements	Base case	Existing case	Design 1	Design 2	Design 3	Design 4	Design 5	Design 6
Pile field spacing (m)	-	7, 7, 7, 14, 20, 14, 16.	6, 6, 10, 10, 10, 14, 14, 15. (-11%)	10, 10, 10, 15, 19, 21. (+14%)	as per existing	as per existing	as per existing	as per existing
Number of pile fields	-	8	9	7	9	9	9	9
Pile height above riverbed (m)	-	1.5	1.5	1.5	1.5	1.5	1	2
Porosity (% open)	-	50%	50%	50%	33%	60%	50%	50%
Pile diameter (mm)	-	300	300	300	300	300	300	300
Stagger piles	-	-	-	-	-	-	Stagger piles with existing spacing	Stagger piles with increased spacing

Table 14. Pile field configuration – St Helens Creek (green indicates variation).

Pile field design parameter	Base case	Existing case	Design 1	Design 2	Design 3	Design 4	Design 5	Design 6
Pile field spacing (m)	-	13, 16, 16, 23, 23, 34, 22, 23.	23, 24, 28, 31, 33, 30. (+29%)	13, 10, 10, 14, 20, 21, 20, 21, 21, 20 (-25%)	as per existing	as per existing	as per existing	as per existing
Number of pile fields	-	9	11	8	9	9	9	9
Pile height above riverbed (m)	-	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Porosity (% open)	-	50%	50%	50%	33%	70%	50%	50%
Pile diameter (mm)	-	300	300	300	300	300	300	300
Groyne angle to upstream flow		45° to flow	45° to flow	45° to flow	45° to flow	45° to flow	90° to flow	65° to flow

Table 15. Pile field configuration – Molongle Creek (green indicates variation).

Pile field design parameter	Base case	Existing case	Design 1	Design 2	Design 3	Design 4	Design 5	Design 6
Pile field spacing (m)	-	10, 8, 11, 8, 21, 20, 22, 21, 21, 21, 20, 17, 22, 19, 21, 19, 19.	As per existing	as per existing	as per existing	as per existing	as per existing	as per existing
Number of pile fields	-	18	18	18	18	18	18	18
Pile height above riverbed (m)	-	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Porosity (% open)	-	50%	33%	60%	50%	50%	50%	50%
Pile diameter (mm)	-	300	300	300	300	300	300	300
Groyne length past toe	-	-	-	-	3	6	-	-
Groyne angle to upstream flow	-	65° to flow	65° to flow	65° to flow	65° to flow	65° to flow	95° to flow	45° to flow

Table 16. Pile field configuration – Yarra River (green indicates variation).

Pile field design parameter	Base case	Existing case	Design 1	Design 2	Design 3	Design 4	Design 5	Design 6
Pile field spacing (m)		20, 7, 13, 10, 8, 5.	10, 13, 13, 15, 11. (15%)	8, 7, 10, 10, 10, 8, 9. (-12%)				
Number of pile fields	-	7	6	8	7	7	7	7
Pile height above riverbed (m)	-	1.5	1.5	1.5	1	2	1.5	1.5
Groyne length past toe of bank (m)	-	0	0	0	0	0	0	0
Porosity (% open)	-	50%	50%	50%	50%	50%	50%	50%
Pile diameter (mm)	-	300	300	300	300	300	300	300
Groyne length past toe	-	-	-	-	-	-	2	4

A summary of the selected sites, and the key pile field design parameters to be assessed at each site, is provided in Table 17. A summary of the key hydro-numeric parameters for each of the seven study sites is provide in Table 18.

Table 17. Summary table of selected sites, and key pile field design parameters to be assessed at each of the seven sites.

Study sites	Pile field spacing (m)	Porosity (% open)	Pile height above riverbed (m)	Groyne length past toe of bank (m)	Staggered Pile fields	Angle
Mary River – Carters	✓	✓	✓	✗	✗	✗
Mary River – Kenilworth	✓	✗	✗	✗	✓	✓
O'Connell River – Site 1	✓	✗	✓	✓	✗	✗
O'Connell River – Site 2	✓	✓	✗	✗	✓	✗
St Helens	✓	✓	✗	✗	✗	✓
Molongle Creek	✗	✓	✗	✓	✗	✓
Yarra River	✓	✗	✓	✓	✗	✗

Table 18. Summary table of key hydro-dynamic parameters for each of the seven study sites.

Site	Reef Catchment	Adjoining Landuse	Construct date	Average bank height (m)	Channel width (m)	Width class	Radius curvature	Ratio RC/width	Sinuosity class
Mary River – Kenilworth Park	Burnett-Mary	Recreation park	2014	8	120	Medium	504	4.2	Low
Mary River – Carters	Burnett-Mary	Grazing	2019	5.5	200	Wide	360	1.8	Medium
O'Connell River Site 1	Mackay Whitsunday	Natural area/grazing	2018	8	88	Medium	-	-	Low
O'Connell River Site 2	Mackay Whitsunday	Cropping	2015	7	145	Narrow	220	1.5	High
St Helens	Mackay Whitsunday	Cropping	2018	5	118	Medium	299	2.5	Medium
Molongle Creek	Burdekin	Agriculture	Design stage	3	104	Medium	196	1.9	Medium
Yarra River	-	Grazing	2008	-	25	Narrow	40	1.6	High

4.4 Model setup

AEM3D

The 3D model used for simulating the pile fields is the Aquatic Ecosystem Model 3D (AEM3D). AEM3D is an updated version of ELCOM-CAEDYM (produced by Centre for Water Research, UWA) that is developed and distributed by Hydronumerics. Technical details for AEM3D can be found at http://www.hydronumerics.com.au/#software.

In terms of the hydrodynamics code, the AEM3D transport equations solve unsteady Reynolds-averaged Navier-Stokes equations and scalar transport equations with the Boussinesq approximation using hydrostatic pressure assumption. The free surface evolution is governed by an evolution equation developed by a vertical integration of the continuity equation applied to the Reynolds-averaged kinematic boundary condition. The equations are solved using the TRIM numerical scheme with modifications to improve accuracy, scalar conservation, numerical diffusion, and implementation of a mixed-layer turbulence closure scheme. Solutions are made on an Arakawa C-grid (orthogonal with option of varying width) in which flow velocity is defined on cell faces and the free-surface height and scalar concentrations are solved at the cell centre. The free-surface height in each column of grid cells moves vertically through the grid to improve computational efficiency and allows sharper vertical gradients to be maintained with coarse grid resolutions.

AEM3D computes solutions at time steps of order of minutes using the following steps:

1. introduce surface heating/cooling in the surface layer;
2. mix scalar concentrations and momentum using a mixed-layer model;
3. introduce wind energy as a momentum source in the wind-mixed layer;
4. solve the free-surface evolution and velocity field;
5. apply horizontal diffusion of momentum;
6. advection of scalars; then
7. horizontal diffusion of scalars.

Heat exchange through the surface is not modelled in the current study.

Resuspension stress (τ_B) is calculated as:

$$\tau_B = \rho\, C_D\, u \sqrt{(u^2 + v^2)}$$

Where ρ is density, C_D is a drag coefficient, and u and v are the velocity components.

Bathymetry grids

Idealised digital elevation models (DEMs) (0.2 m resolution) for the sites were developed from LiDAR datasets for input into the 3D model (Figure 33 to Figure 38). These idealised DEMs were developed with the aim to reduce “noise” in model outputs (i.e. bed shear stress) caused by artifacts in the raw terrain models. The idealised channel captures key geomorphic features of the site, including bank height, channel width and sinuosity, geomorphic units (i.e. benches), and reach slope. The DEMs incorporated a reprofiled bank (to an approximate gradient of 1V:3H) (excluding the Yarra River Site); to ensure the impact of pile fields design parameters, rather than the bank morphology, was being assessed. Shapefiles of the pile field configurations were then developed for input into the AEM3D model, with the elevation (m AHD) of the top of each individual pile assigned to each pile polygon. For the pile field scenarios pile fields were built into the DEMs.

AEM3D bathymetric grids were generated from the idealised DEMs and shapefiles of the existing and design pile field locations. To resolve the 30 cm pile field grids a 10cm horizontal grid in the region of the pile field was required. The horizontal grid size was then increased gradually out to a resolution of 5 m away from the pile field. Pile field cells are set to the height of the pile when the centre of the grid cell is within the circular polygon shape that defines the pile in the shapefile. Example plots for the Mary River Carters site are shown in Figure 30 and Figure 31.

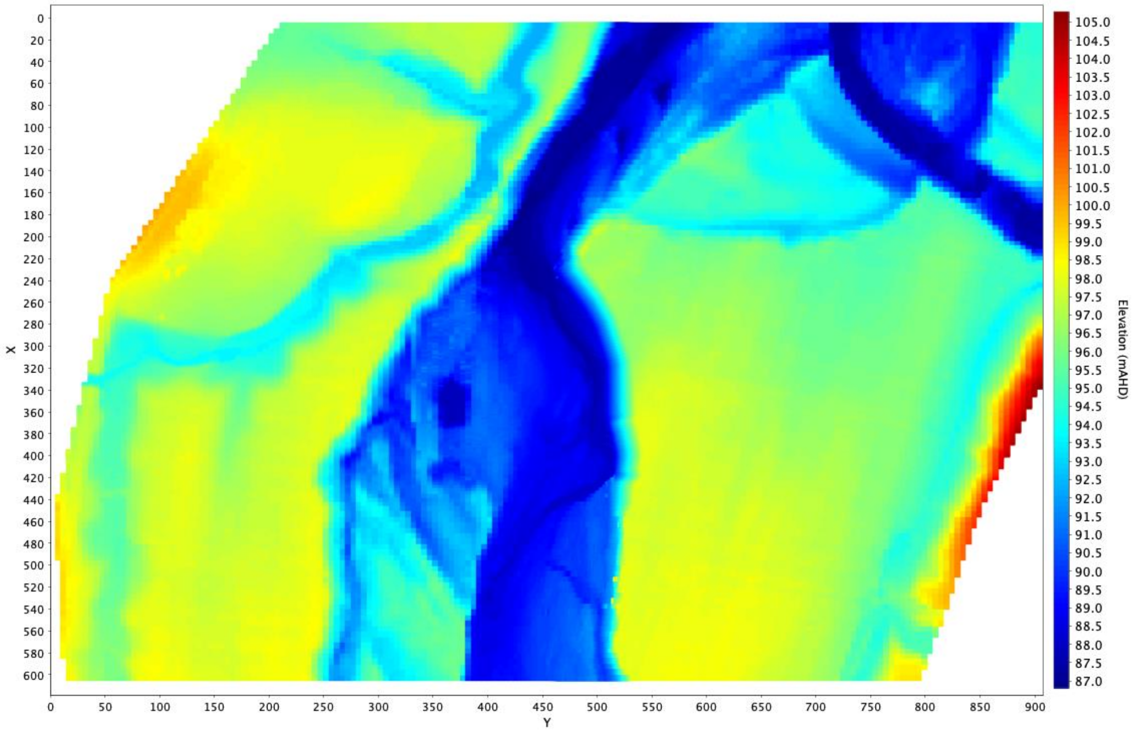

Figure 30. *Full AEM3D bathymetric grid for Mary River Carters domain.*

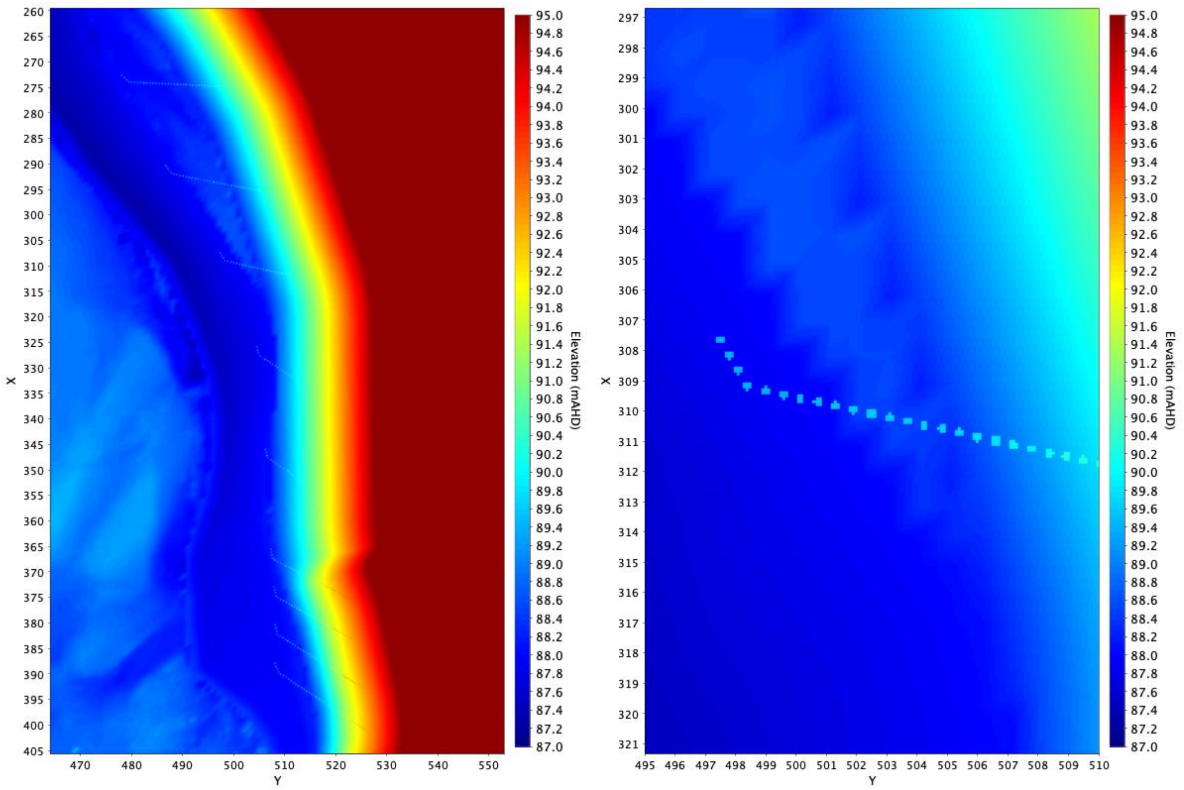

Figure 31. *Close ups of the pile field region of the Mary River Carters bathymetric grid.*

Model setup

Each model domain was configured with an upstream inflow boundary condition and a downstream open boundary condition. On the upstream inflow boundary condition, a constant flow was specified and on the downstream open boundary condition, a constant height specified. Manning's n regions were delineated based on analysis on high resolution aerial imagery.

At each site, three design flows were identified to assess the hydraulic effectiveness of pile fields under various flow elevations relative to pile and bank height:

1. flow height at the top of the riverward piles (i.e. flow height 1.5 m above the bed),
2. flow approximately twice the height of the riverward piles (i.e. flow height approximately 2.5 – 3 m above the bed), and
3. flow overtopping the landward piles (all piles submerged).

Exemplar design flows developed for the Mary River Carters site are shown in Figure 39.

The model timestep for all domains was set to 1 second. For each domain the following steps were undertaken:

1. A no pile field model was initialised with a constant surface level and run for 2 hours of simulation time to allow the free surface gradient to be setup and stabilise;
2. The final simulated height from the no pile field initial run was used as the initial surface level for each of the pile field designs (including a rerun of the no pile field case);
3. Each design configuration for each site and each flow rate was run for 30 minutes to allow the system to stabilise (i.e. to reach "steady-state");
4. Processing of the resuspension stresses was undertaken using the average of the final 10 minutes of each simulation (to eliminate discrepancies in flow oscillations between the pile field and no piles field scenarios).

HECRAS 2D
2D hydrodynamic simulation for this study were performed using HEC-RAS (version 6.0); using the Shallow Water Equations (SWE). For the 2D cells shear stress is the average shear stress across each face, then interpolated between face.

Shear stress (τ) is calculated as:

$$\tau = \gamma R S_f$$

Where γ is the unit weight of water, R is the hydraulic radius, and S_f is the friction slope of water.

As far as possible, the 3D modelling procedures were replicated for the 2D model setup; to enable comparison of 3D and 2D model results. RASmapper was used for the HEC-RAS 2D model set up. The idealised terrain model, developed for AEM3D, was input into the 2D model. The pile fields were then built into the into the terrain.

A 2D computational mesh was used to delineate the model domain, with a base resolution of 5 m by 5 m, and a higher resolution adjacent to the piles fields (0.1 m by 0.1 m). The cell size was gradually increase from 0.1 m adjacent to the pile fields, out to a resolution of 5 m. For each design flow an unsteady flow simulation was performed, however, the peak flow was run for an extended period (1-2 hours) to simulate a "steady-state". The model used a variable time step, with a minimum computational interval of 0.1 – 0.5 seconds.

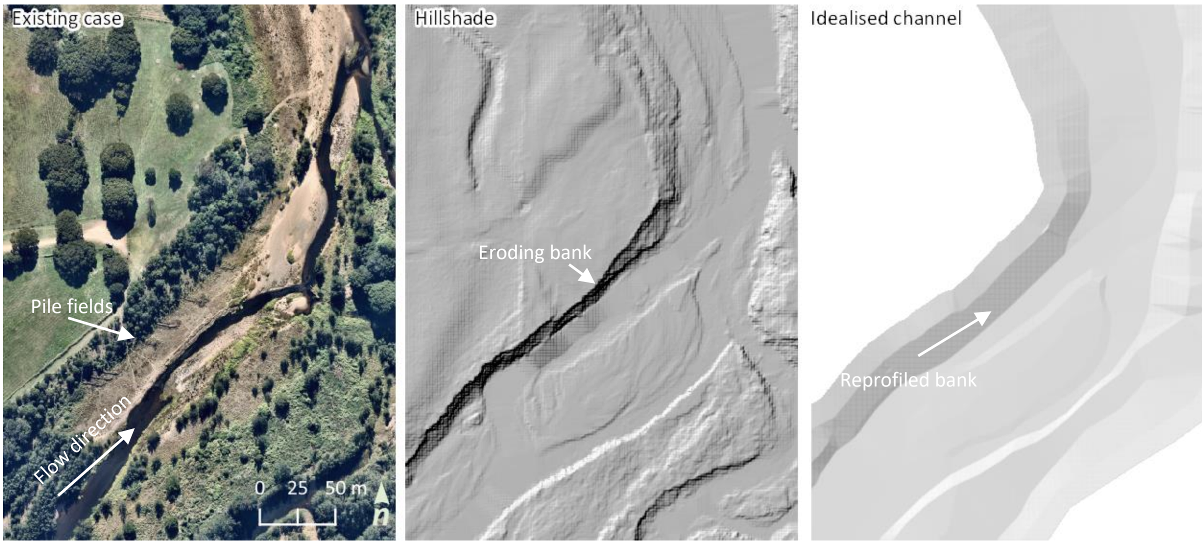

Figure 32. *Mary River Kenilworth site showing the existing pile field design (left), hillshade of the pre-works terrain (middle), and the of35alized terrain model*.

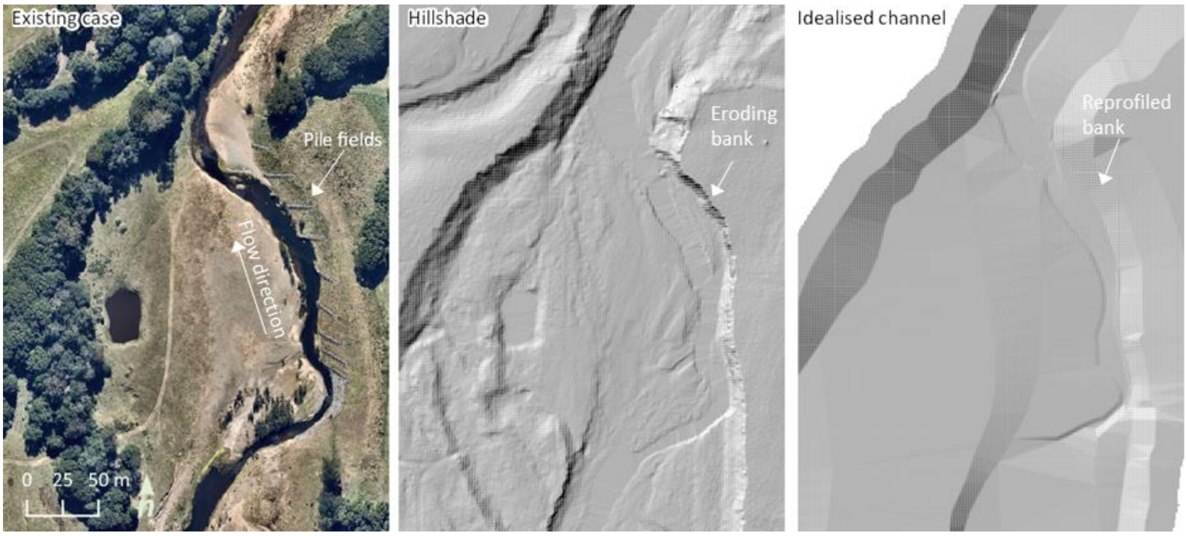

Figure 33. *Mary River Carters site showing the existing pile field design (left), hillshade of the pre-works terrain (middle), and the of35alized terrain model*.

Figure 34. *O'Connell River Site 1 showing the existing pile field design (left), hillshade of the pre-works terrain (middle),*

*and the of36alized terrain model***.**

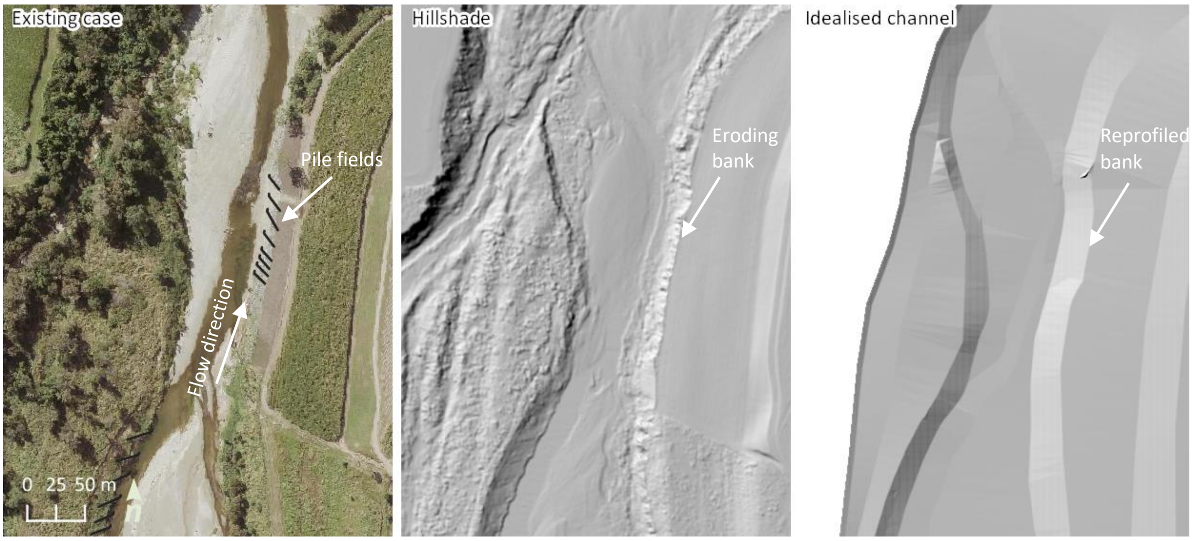

Figure 35. *O'Connell River Site 2 showing the existing pile field design (left), hillshade of the pre-works terrain (middle), and the of36alized terrain model***.**

Figure 36. *St Helens Creek site showing the existing pile field design (left), hillshade of the pre-works terrain (middle), and the of36alized terrain model***.**

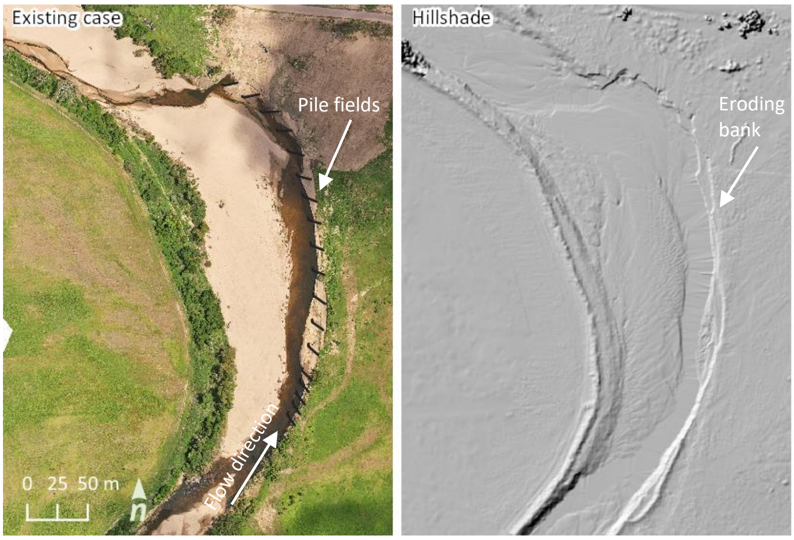

Figure 37. *Molongle River site showing the existing pile field design (left), hillshade of the pre-works terrain (middle), and the of37alized terrain model***.**

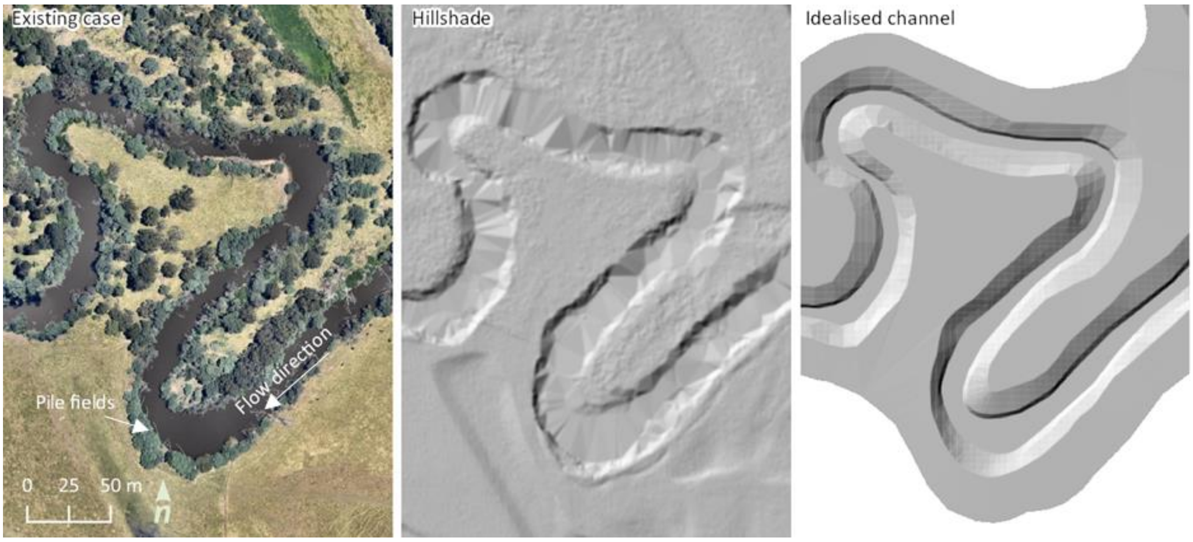

Figure 38. *Yarra River site showing the existing pile field design (left), hillshade of the pre-works terrain (middle), and the of37alized terrain model***.**

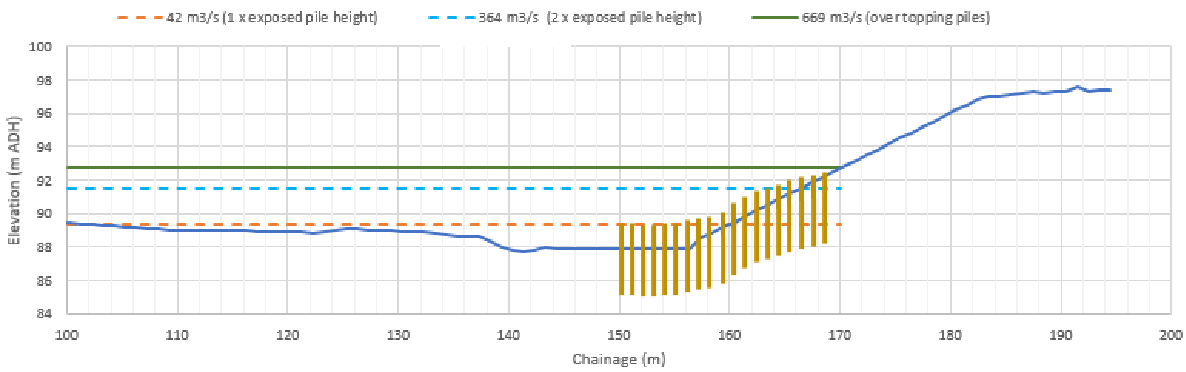

Figure 39. *Cross-section showing the three flows modelled at Mary River (Carters).*

5 Results

The 3D and 2D models use different bed shear stress equations (as outlined in Section 4.3). The percentage change in bed shear stress was considered of most useful comparator between the two modelling approaches. The 3D and 2D percentage change in bed shear stress equations are provided below.

Energy loss, stream power and shear stress *(From Tilleard, 1999)*
Water can have potential energy by virtue of its elevation. As water flows within a channel this potential energy is converted into kinetic energy of moving water and any debris and sediment load that is carried. Energy is dissipated as the result of turbulence within the flow and by friction between the flow (and any load) and the channel boundaries. ***Stream power*** *is the rate of energy expenditure.*

In a channel, at least part of the energy expenditure occurs as a result of the ***velocity gradient*** *in the flow close to the boundaries. This shearing of flow close to the boundary occurs as a result of the force exerted on the flow by the irregularities associated with the boundaries at a particle and a bedform scale. Per unit area of boundary, this force is the* ***average boundary shear stress****.*

Shear stress *is the force exerted against the channel and floodplain boundary during flow events. Once the critical shear stress value (for sediment mobilisation) is reached the channel boundary material may begin to erode (mobilise).*

3D percentage change in bed shear stress is calculated as:

$$\left(\frac{(Mean\ resuspension\ stress\ (pile\ field\ scenario) - Mean\ resuspension\ stress\ (no\ pile\ field\ scenario))}{Mean\ resuspension\ stress\ (no\ pile\ field\ scenario)}\right) \times 100$$

2D percentage change in bed shear stress is calculated as:

$$\left(\frac{(Shear\ stress\ (pile\ field\ scenario) - Shear\ stress\ (no\ pile\ field\ scenario))}{Shear\ stress\ (no\ pile\ field\ scenario)}\right) \times 100$$

Both the no pile field and the pile field scenarios incorporated a reprofiled bank (to an approximate gradient of 1V:3H) (excluding the Yarra River Site). Therefore, the shear stress reduction due to pile field bank stabilisation (and associated bank reprofiling) compared to eroding bank conditions (i.e. vertical banks) is likely greater than results indicate

Box plots of the variation in near-bank (i.e. within the pile field embayments) percentage change in bed shear stress for the design configurations at each site are shown in Figure 40 to Figure 46. The top plots show the results of the 3D simulations, and the bottom plots show the results of the 2D simulations. Spatial distribution of the percentage change in bed shear stress for the alternative design scenarios for each site modelled in AEM3D and HEC-RAS 2D is provided in Attachment B.

The box plot interquartile range (IQR) is a visual representation of the range (of percentage change in bed shear stress) from the first to the third quartile. The upper whisker extends from the top of the box to the largest data element that is less than or equal to 1.5 times the IQR. The lower whisker extends from the bottom of the box to the smallest data element that is larger than 1.5 the IQR. The 3D shear stress results were averaged over a 10-minute period at peak flow to eliminate discrepancies in flow oscillations between the pile field and no piles field scenarios. Where the bed shear stress for the no pile field scenario was very small compared to the existing pile field scenario the percentage change was very high in isolated cells (i.e >2500%). Therefore, for clarify, outliers are not shown on the boxplots.

A summary of results for each pile field design parameter assessed is provided below.

Groyne porosity
The 2D and 3D simulations show a similar tendency with the pile fields resulting a greater reduction bed shear as groyne porosity decreases (from 50% to 33% open) (Figure 41, Figure 43, Figure 44, and Figure 45). Reduction in shear stress, due to a decrease in porosity, was generally slightly more notable in the low flow scenarios. At the Molongle River site the existing scenario (50% porosity) (low flow) had a median change in bed shear stress of -68% and -62% for the 2D and 3D simulations respectively. Reducing porosity to 33% results in an additional 12% and 14% reduction. Comparative reductions, due to reduction in porosity, were evident at the St Helens Creek site.

Spacing between groynes
The 2D and 3D simulations indicate that reducing the spacing between pile fields results in moderate reduction in bed shear (Figure 40, Figure 41, Figure 42,, Figure 43, Figure 45, and Figure 46). Reduction in shear stress, due to a decrease in pile field spacing, was relatively consistent across low, mid and high flow scenarios. Decreasing pile field spacing by 25% at St Helens Creek only resulted in a -%5 and -6% change in bed shear stress in the 2D and 3D low flow scenarios respectively. A slightly larger decrease was evident at Mary River Kenilworth where decreasing pile field spacing by 26% resulted in a -%10 and -11% change in bed shear stress in the 2D and 3D low flow scenarios respectively.

The exposed height of piles (above the riverbed)
The 3D and 2D results show that pile fields result in a greater reduction in bed shear stress as the exposed height of the pile increased (Figure 41, Figure 42, and Figure 46). Reduction in shear stress, due to an increase in the exposed height of the pile fields, was generally most notable in the mid and high flow scenarios. At O'Connell River site 1 the existing scenario (pile 1 .5 m above riverbed) (high flow) had a median change in bed shear stress of -26% for both the 2D and 3D simulations, compared with a significant (-46%) reduction for the pile 2.5 m above riverbed scenario.

Groyne length past toe of bank (into channel)
The 3D simulations show that groyne length past toe of bank had a variable impact on shear stress reduction. (Figure 42, Figure 44 and Figure 46). Extending the groyne past the toe of bank generally resulted in a slightly lesser reduction in near bank shear stress in low flows, and generally greater reduction in shear stress in mid and high flows. However, this trend was not consistent across all three sites. Comparatively, the 2D simulations show that extending the groyne length past toe of bank results in a lesser reduction in bed shear stress across all three sites, under all three flow scenarios. For the Molongle Creek site the existing scenario (groyne 3 m past bank toe) (low flow) had a median change in bed shear stress of -68% for the 2D simulations, compared with -62% and -57% for the 6 m and 9 m groyne length past bank toe scenarios respectively. Additionally, the low flow scenarios, have higher interquartile range compared with the mid and high flow scenarios, suggesting higher variability.

Groyne angle relative to upstream flow
The 2D and 3D simulations show that groyne angle relative to flow direction had marginal impact on shear stress reduction (Figure 40, Figure 44 and Figure 45). At the Mary River Kenilworth site adjusting the groyne angle from 90^o to 45^o (relative to upstream flow) resulted in a +%6 and +4% change in bed shear stress in the 3D and 2D low flow scenarios respectively. However, at mid and high flows reducing groyne angle relative to flow resulted in a marginal decrease in bed shear stress (approximately -2 to -6%) at the Mary River Kenilworth site for both 2D and 3D simulation.

Stagger piles (i.e. each pile field comprising two stagger rows)
The 2D and 3D simulations show that staggering the pile field generally results in a considerable reduction in shear stress (Figure 40 and Figure 43), except for the 3D O'Connell River Site 2 low flow simulation which results in an increase in shear stress. However, this may be a result of the water level being close to where there is a drop in the bathymetry which also coincides with the gradient the model grid. For the Mary Kenilworth site the existing scenario (medium flow) had a median change in bed shear stress of -43% and -42%

for the 2D and 3D simulations respectively, compared with -66% and -58% for staggered (with existing design) scenarios. This represents a 53% and 36% overall decrease in shear stress relative to the existing design.

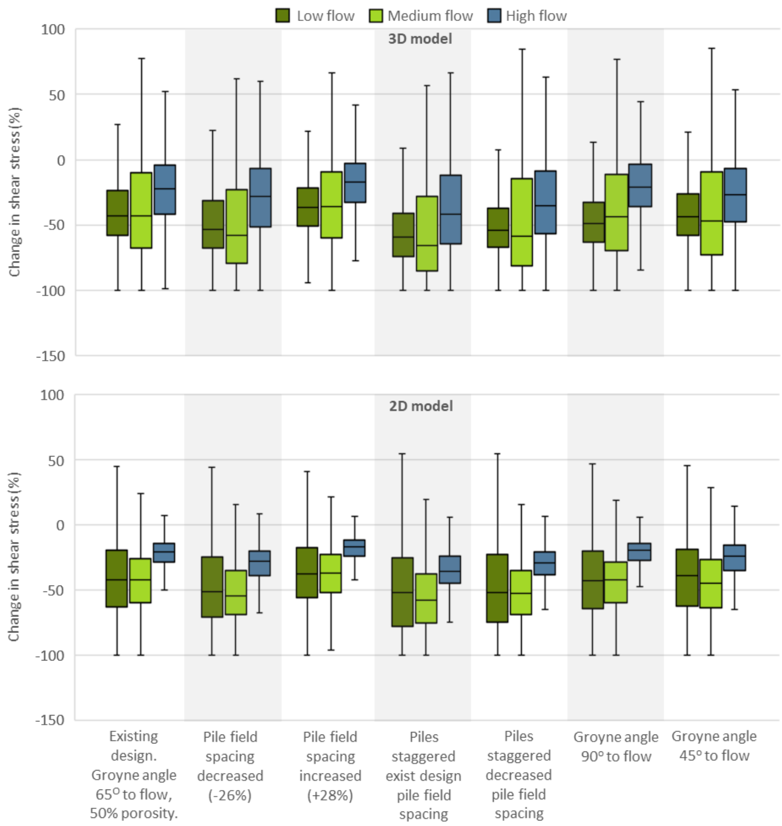

Figure 40. *Box-and-whisker plot showing variation in percentage change in bed shear stress between the Mary River Kenilworth design scenarios and no pile field scenario for the three flows simulated. AEM3D (top), HECRAS2D (bottom).*

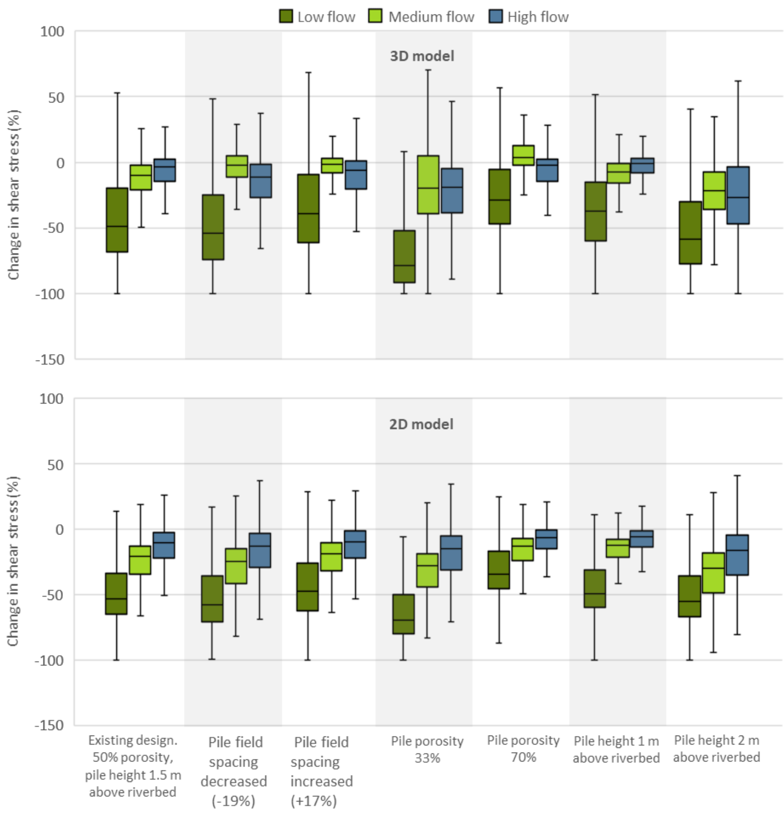

Figure 41. *Box-and-whisker plot showing variation in percentage change in bed shear stress between the Mary River Carters design scenarios and no pile field scenario for the three flows simulated. AEM3D (top), HECRAS2D (bottom).*

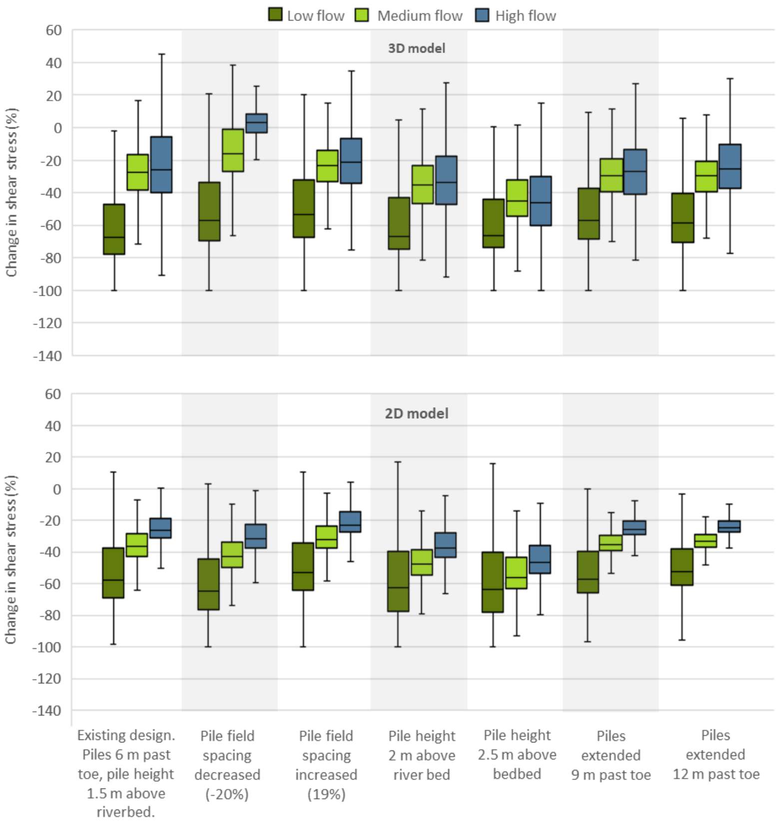

Figure 42. *Box-and-whisker plot showing variation in percentage change in bed shear stress between the O'Connell River Site 1 design scenarios and no pile field scenario for the three flows simulated. AEM3D (top), HECRAS2D (bottom).*

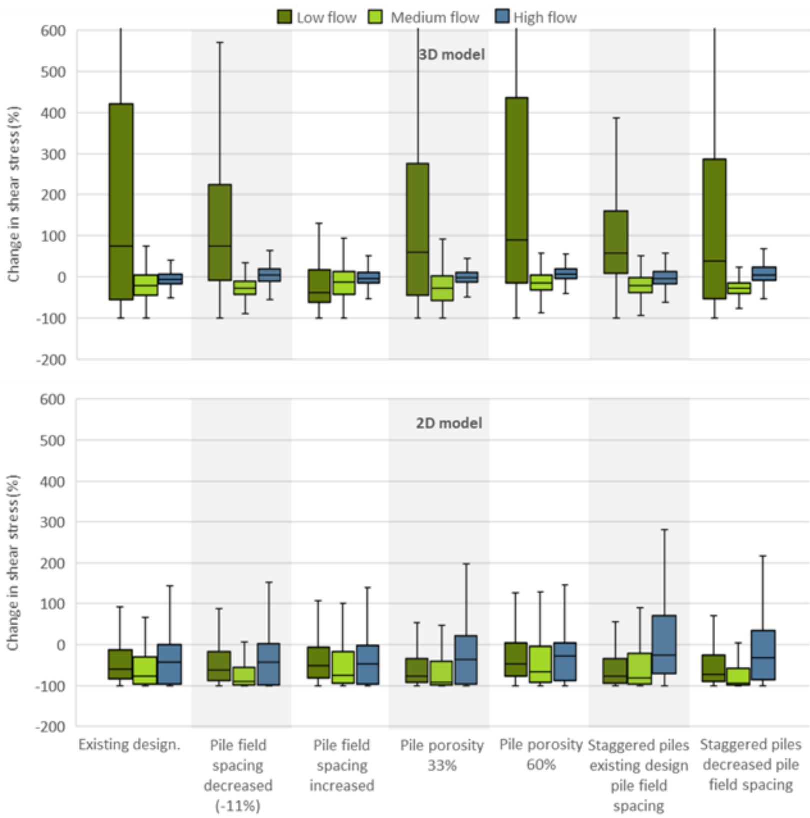

Figure 43. *Box-and-whisker showing variation in percentage change in bed shear stress between the O'Connell River Site 2 design scenarios and no pile field scenario for the three flows simulated. AEM3D (top), HECRAS2D (bottom).*

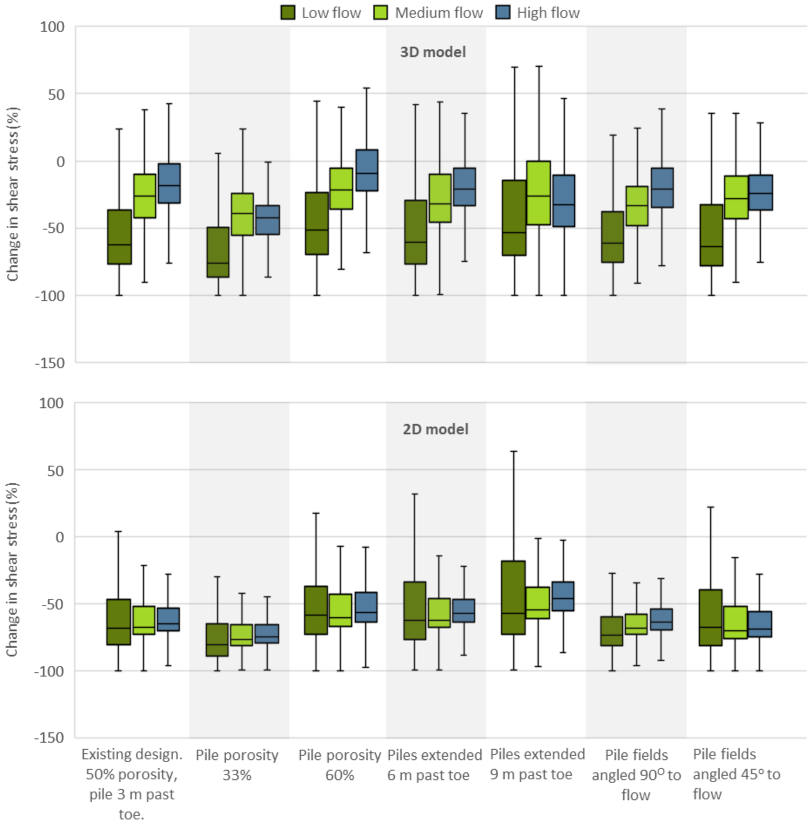

Figure 44. *Box-and-whisker showing variation in percentage change in bed shear stress between the Molongle Creek design scenarios and no pile field scenario for the three flows simulated. AEM3D (top), HECRAS2D (bottom).*

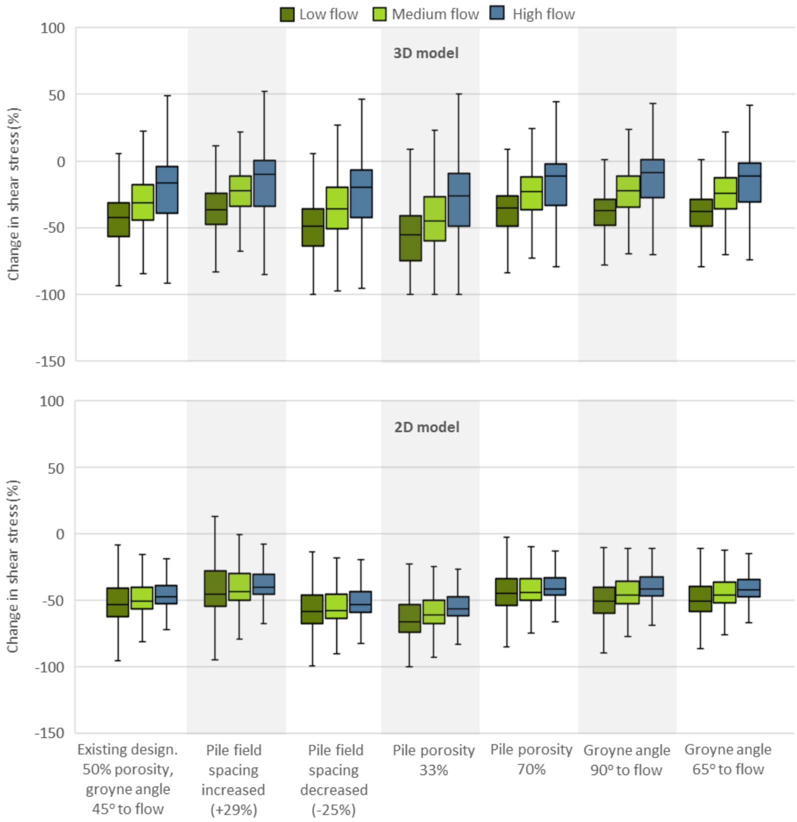

Figure 45. *Box-and-whisker showing variation in percentage change in bed shear stress between the St Helens Creek design scenarios and no pile field scenario for the three flows simulated. AEM3D (top), HECRAS2D (bottom).*

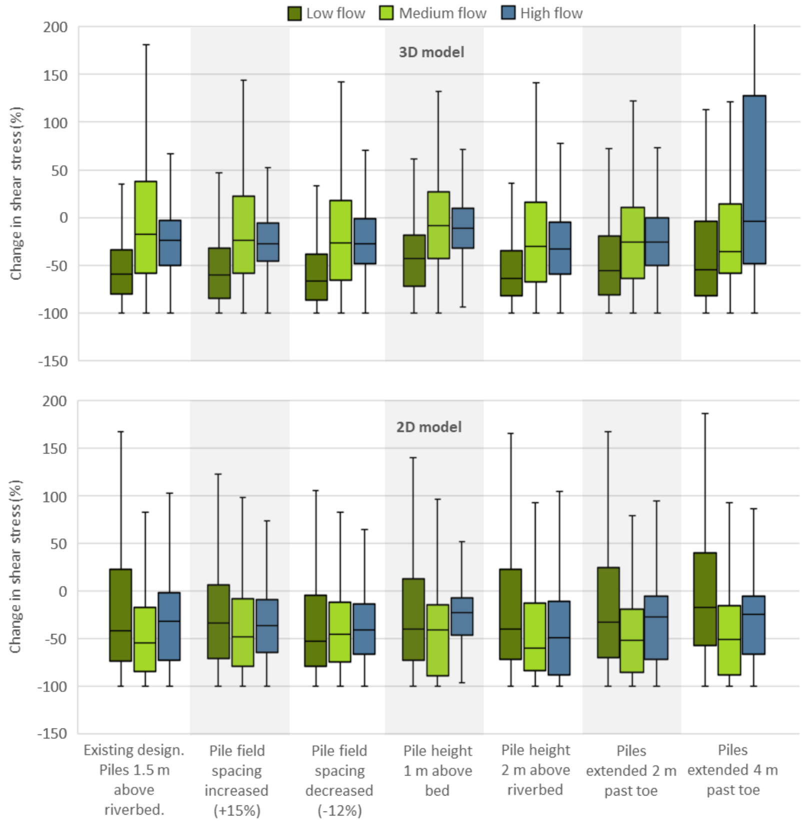

Figure 46. *Box-and-whisker plot showing variation in percentage change in bed shear stress between the Yarra River design scenarios and no pile field scenario for the three flows simulated. AEM3D (top), HECRAS2D (bottom).*

6 Discussion

6.1 Overview

Hydraulic modelling shows that pile fields impact spatial velocity distribution across the river channel. Pile fields result in a reduction in near-bank velocity (within pile field embayments) and an increase in flow velocity in the main channel (Figure 47). Pile fields also prevent bank-directed flow downstream of the structures.

As expected, modelling indicates that the pile field groynes are generally more effective during low (frequent) flow events when the pile fields are fully engaged (i.e. not overtopped). Interestingly, the 2D simulations generally indicate a greater reduction in bed shear than the 3D model for all flow events.

A summary of the hydraulic impacts of pile field design variables, and influence of hydrogeomorphic features, is provided below.

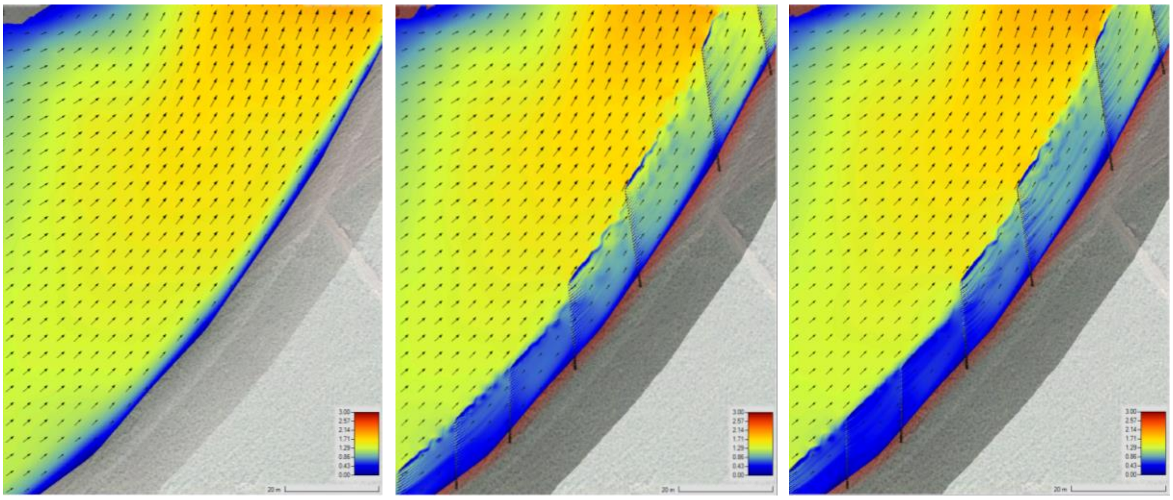

Figure 47. *Distribution of the spatial velocity (m/s) at the St Helens Creek site (Low flow, 2D model). No pile field scenario (left), existing pile field design 50% porosity (middle), and decreased porosity scenario 33% (right).*

6.2 Groyne porosity

Reducing or increasing groyne porosity has a significant impact on near-bank shear stress. At the St Helens Creek site decreasing porosity from 50% to 33% open resulted in a 30% decreased in percent change in shear stress (Figure 48). Whereas increase porosity resulted in a 17% relative increase. However, reducing porosity (below 40%) is correlated with an increase in erosive downward flows (Kang et. al., 2011). Therefore, design should consider the impact of porosity on both near bank velocity and scour.

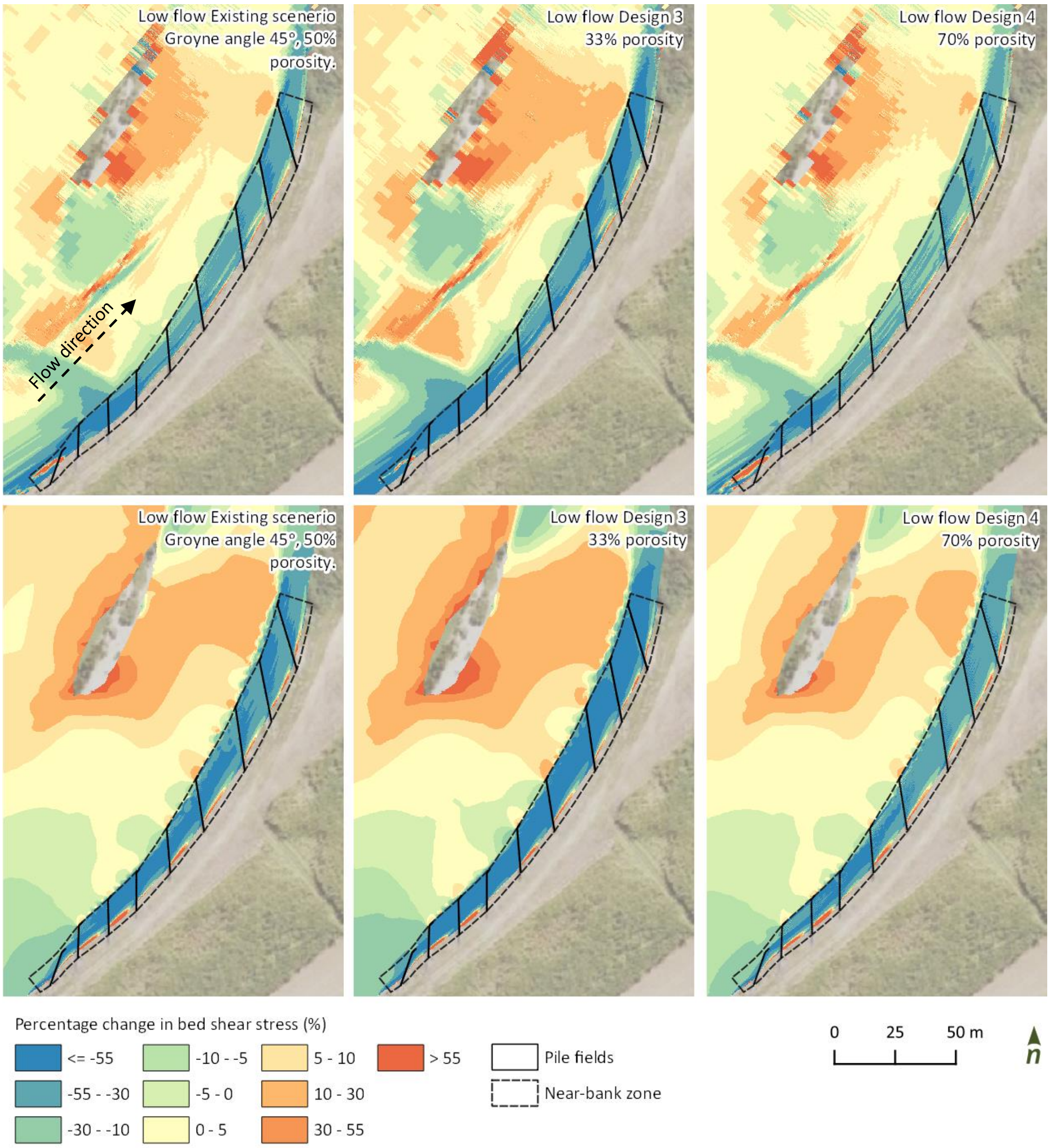

Figure 48. *Impacting of porosity on percentage change in bed shear stress between the pile field scenarios and no pile field scenario showing (St Helens Creek site). AEM3D (top row), and HECRAS 2D (bottom row).*
As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase.

6.3 Groynes spacing

Decreasing and increasing pile field spacing had moderate impact on pile field performance. Figure 49 shows a moderate increase in shear stress reduction as the spacing between pile fields decreases at the St Helens Creek site; where decreasing pile field spacing by 25% only resulted in a -5% and -6% change in bed shear stress in the 2D and 3D low flow scenarios respectively.

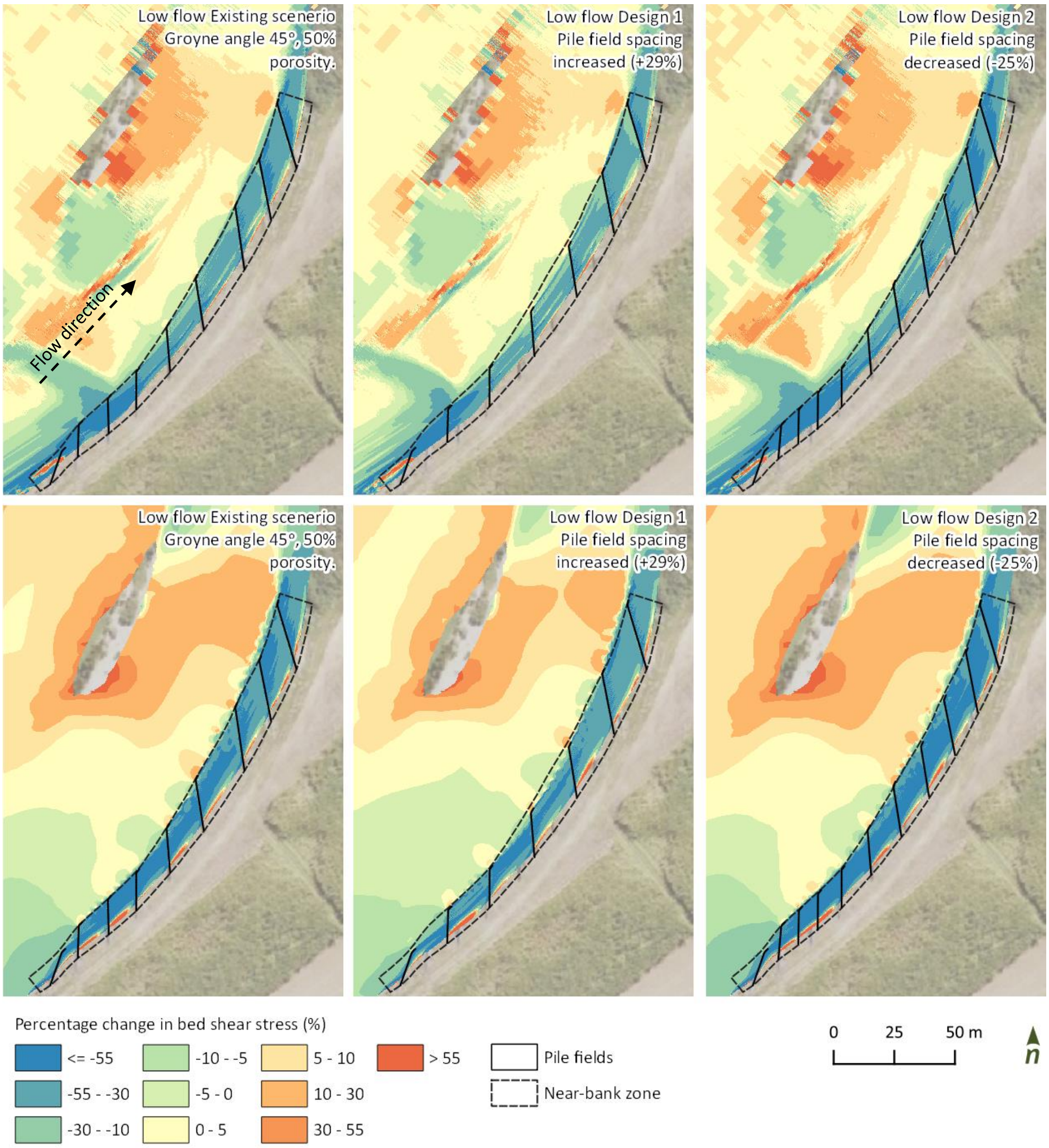

Figure 49. *Impacting of pile field spacing on percentage change in bed shear stress between the pile field scenarios and no pile field scenario showing (St Helens Creek site). AEM3D (top row), and HECRAS 2D (bottom row).*
As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase.

6.4 Pile height

Increasing or decreasing the exposed pile height has significant impact on near-bank shear stress. Figure 50 shows that the higher shear stress reduction zones (indicated by dark blue) expand within the pile field embankments as the exposed pile height increases from 1.5 m to 2.5 m. In general pile field are more effective during low (frequent) flow events when the pile fields are fully engaged. The modelling results show that increasing the exposed pile height improves pile performance in higher flow events.

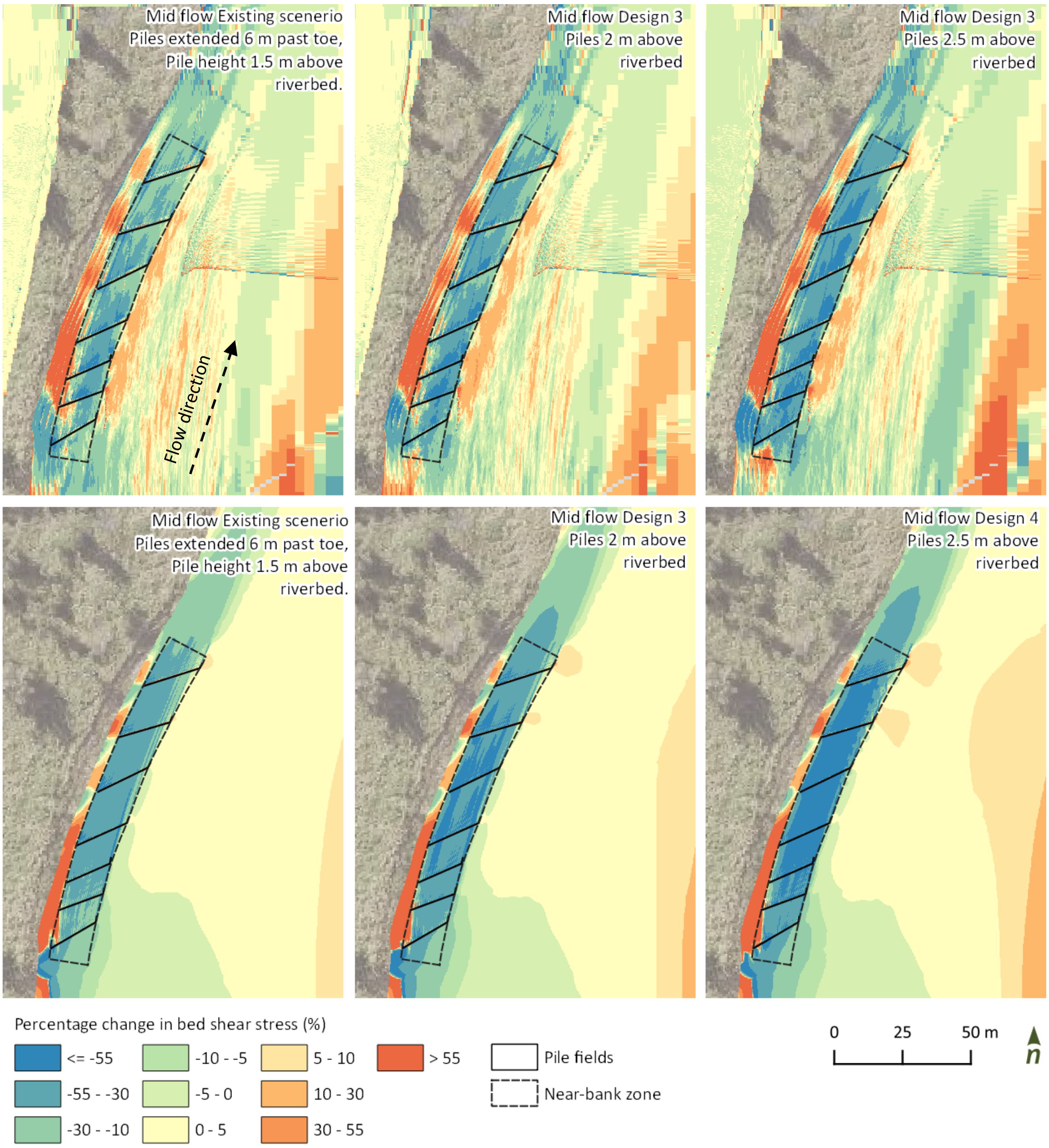

Figure 50. *Impacting of exposed pile height above riverbed on percentage change in bed shear stress between the pile field scenarios and no pile field scenario showing (O'Connell River Site 1). AEM3D (top row), and HECRAS 2D (bottom row). As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase.*

6.5 Groyne length past toe

Increasing the groyne length past the bank toe does not have significant impact on near-bank shear stress. However, simulations show that a higher shear stress band generally occurs at the riverward end of the pile fields. By extending the groynes past the toe of bank this high shear stress band is pushed into the channel and a low shear stress band develops along the toe of bank, this is particularly evident in the Molongle Creek 3D simulations (Figure 51). While extending the groyne length past the toe of bank does not have significant impact on near-bank shear stress, it does provide additional protection (i.e. shear stress reduction) along the reprofiled toe of bank.

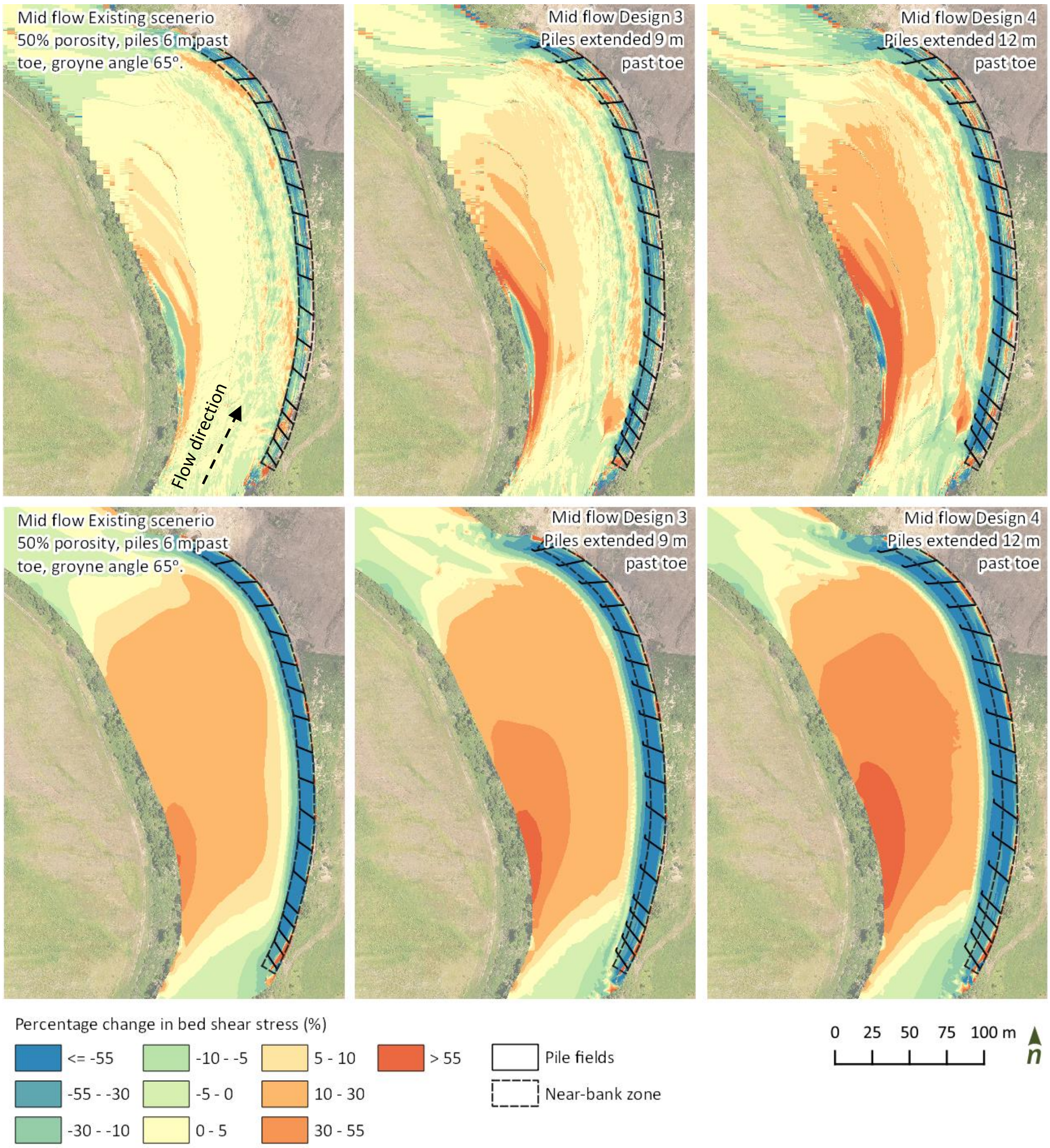

Figure 51. *Impacting of pile field spacing on percentage change in bed shear stress between the pile field scenarios and no pile field scenario showing (Molongle Creek site). AEM3D (top row), and HECRAS 2D (bottom row).*
As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase.

6.6 Groyne angle

Increasing or decreasing groyne angle relative to upstream flow has marginal impact on near-bank shear stress. Reducing the pile field angle from 65° to 45° at the Molongle Creek site (mid flow) had marginal impact on bed shear stress in the 2D and 3D simulations (-2 and -3 % reduction respectively) (Figure 52). In contrast, increasing the pile field angle from 65° to 90° resulted in a -7% reduction in the 3D simulation. Therefore, the results of this study do not support the existing Notional line of attach design approach.

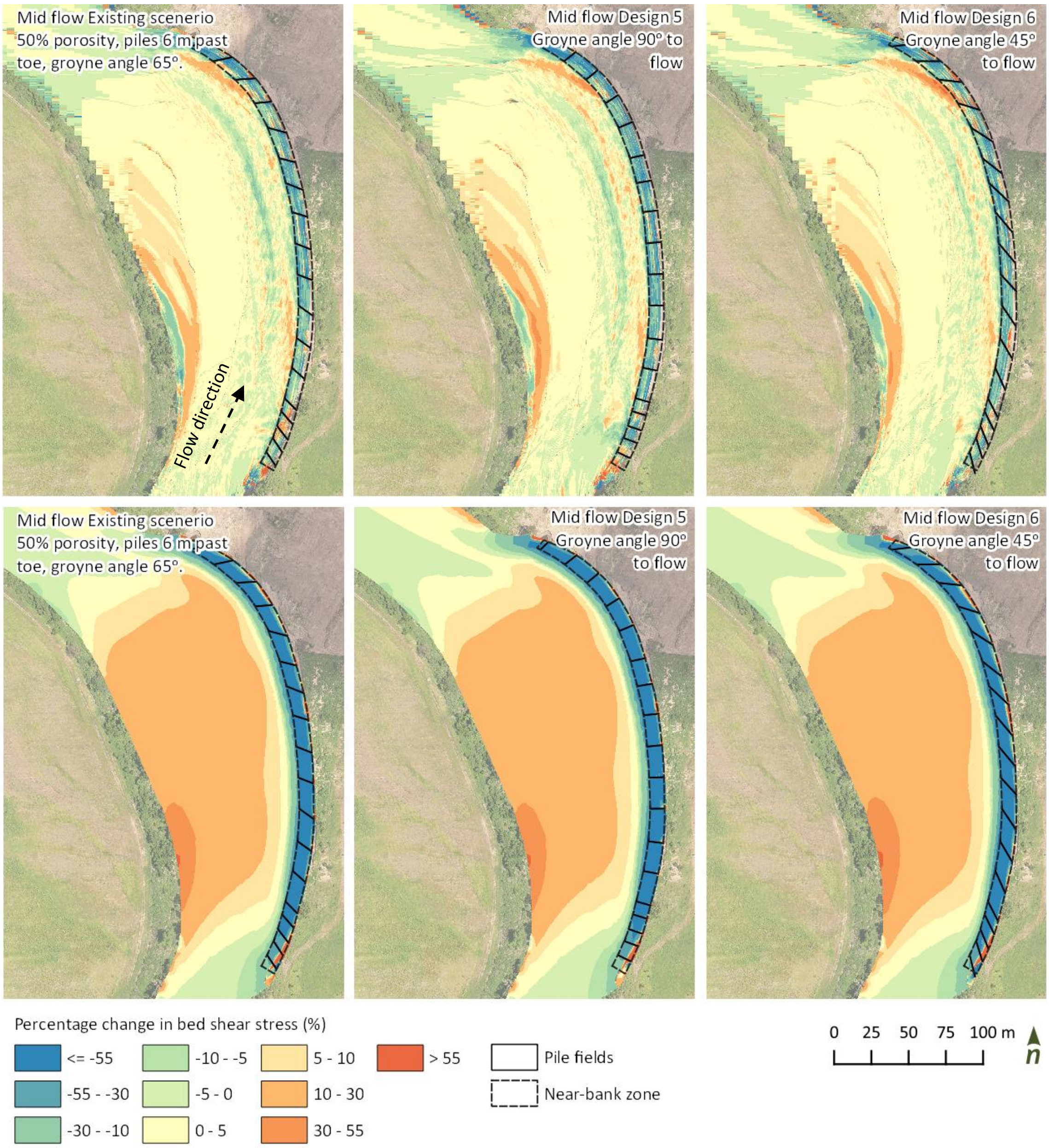

Figure 52. *Impacting of groyne angle on percentage change in bed shear stress between the pile field scenarios and no pile field scenario showing (Molongle Creek site). AEM3D (top row), and HECRAS 2D (bottom row).*
As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase.

6.7 Stagger piles (i.e. each pile field comprising two stagger rows)

Staggering the pile fields results in a significant reduction in bed shear stress (relative to the existing design). At the Mary Kenilworth site staggering the pile fields (with existing design spacing) resulted in a 53% and 36% overall decrease in shear stress relative to the existing design for the 2D and 3D scenarios respectively (Figure 53). However, staggering the piles requires twice as many piles, and is therefore associated with considerably higher construction cost. The staggered pile field arrangement may therefore only be suitable across a relatively small bank stabilisation site where an asset is at risk.

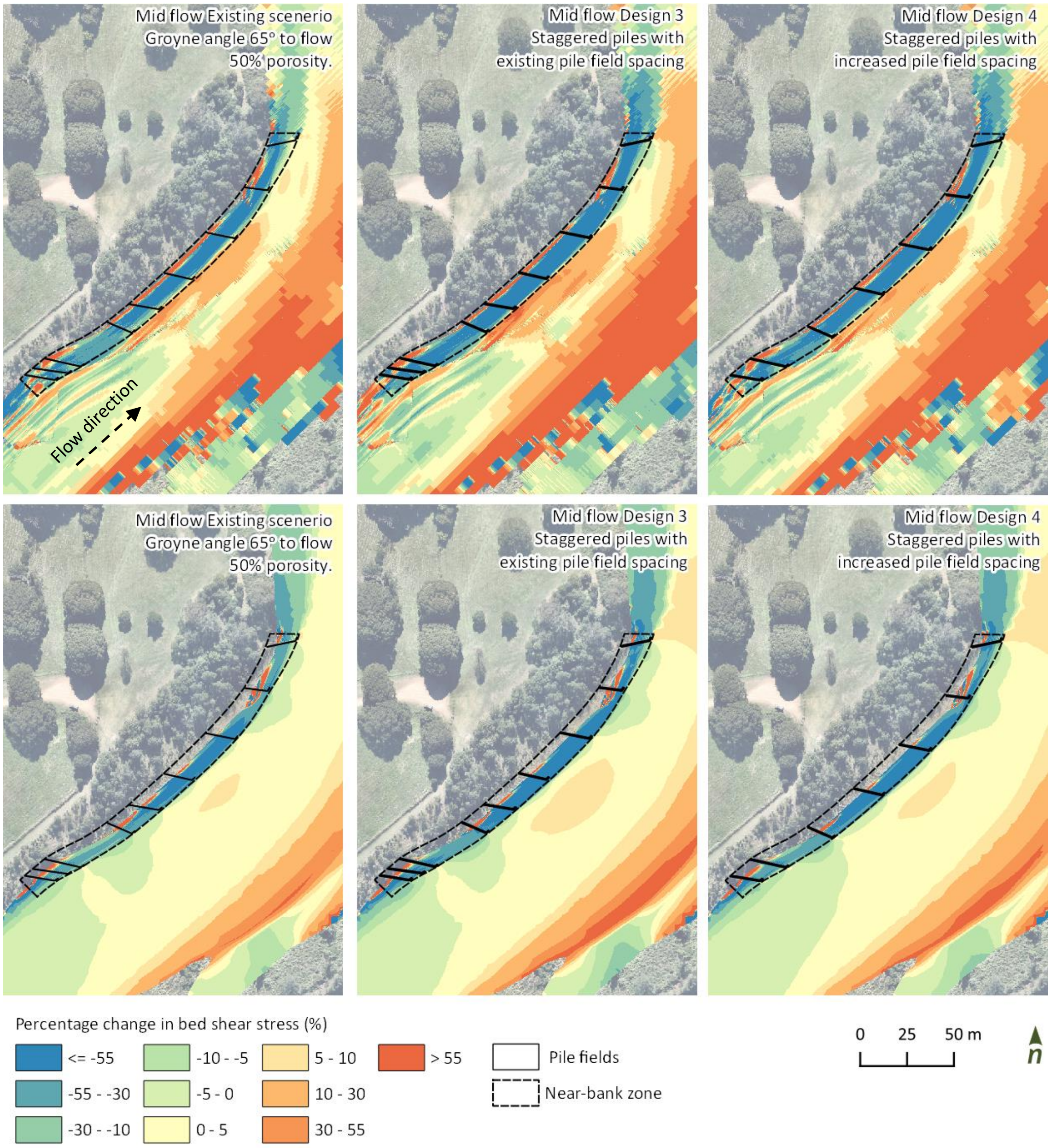

Figure 53. *Impacting of staggering piles on percentage change in bed shear stress between the pile field scenarios and no pile field scenario showing (Mary River Kenilworth site). AEM3D (top row), and HECRAS 2D (bottom row).*
As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase.

6.8 Impact of river morphology

The project aimed to test the influence of river morphology on pile field performance. However, it is difficult to draw conclusive findings as only seven sites were assessed, all with unique hydro-geomorphic characteristics and with many interacting variables. Key observations include:

- The reduction in shear stress due to decrease in spacing was more evident at the Mary River Kenilworth site which is characterised by a low sinuosity planform (Figure 40), compared to the highly sinuous Yarra River site (Figure 46).

- Decreasing porosity (% open) had a greater relative shear stress reduction at the Mary River Carters which is characterised by a wide channel (200 m wide)(Figure 42), compared to St Helens Creek and Molongle Creek which have medium width channels (100 – 120 m wide)(Figure 44 and Figure 45).

6.9 Pile field bank stabilisation costing

An understanding of the impact of pile field design variation and relative shear stress reduction can inform cost effective design. Three key factors inform cost of pile field bank stabilisation works:

- Pile supply cost (dependent on individual pile length)
- Cost to pitch each pile (based on number of piles)
- Cost per driven/embedment depth (based on individual pile length and design flow)

Table 20 provides a summary of the relationship between average shear stress reduction (2D and 3D results) and relative cost of bank stabilisation for the design variations at the seven sites.

The cost impact (percent change) due to variation in pile field design is based on the number of piles and total length of piles relative to the existing design. Pile length assumptions using to determine the cost impact are provide in Table 19.

Table 19. Pile height above riverbed relative to individual pile height.

Pile height above riverbed (m)	Individual pile length (m)
1	4.5
1.5	6
2	7.5
2.5	9

Table 20. Summary of the relationship between average shear stress reduction and relative cost of pile field bank stabilisation. Greens indicate greater reduction in shear stress relative to existing design (darker greens indicate greater reduction). Blue and red indicates decrease and increase in cost respectively.

	Low flow (1 .5 - 2 m above riverbed)	Mid flow (3 – 3.5 m above riverbed)	High flow (over topping pile fields)	Low flow (1 .5 - 2 m above riverbed)	Mid flow (3 – 3.5 m above riverbed)	High flow (over topping pile fields)	Number of piles (% change)	Total length piles (% change)	Cost impact (% change)
	3D simulations			2D simulations			Relative to existing		
Existing (1.5 m above riverbed, 50% porosity)	-54	-25	-17	-54	-50	-35			
Decreased pile field spacing (-26 to -12%)	-55	-27	-13	-55	-53	-34	16%	16%	16%
Increased pile field spacing (15 to 29%)	-47	-21	-14	-48	-42	-30	-14%	-14%	-14%
Pile porosity 33%	-66	-33	-19	-74	-65	-45	29%	29%	29%
Pile porosity 60-70%	-29	-14	-4	-47	-47	-33	-25%	-25%	-25%
Groyne angle 90° to flow (+45°)	-38	-23	-9	-50	-46	-41	-32%	-32%	-32%
Groyne angle 65° to flow (+25°)	-38	-24	-11	-50	-46	-42	-24%	-24%	-24%
Groyne angle 90° to flow (+25°)	-49	-44	-21	-43	-43	-20	-14%	-14%	-14%
Groyne angle 45° to flow (-20°)	-43	-47	-27	-39	-45	-24	21%	21%	21%
Pile height 1 m above riverbed	-40	-8	-6	-45	-27	-14	0%	-25%	-25%
Pile height 2 m above riverbed	-63	-29	-31	-53	-46	-34	0%	25%	22%
Pile height 2.5 m above riverbed	-67	-45	-46	-64	-56	-46	0%	50%	45%
Stagger piles existing design spacing	-59	-44	-23	-65	-70	-31	93%	93%	93%
Stagger piles decreased spacing	-54	-89	-30	-126	-146	-62	116%	116%	116%
Groyne extended 3-4 m past existing	-58	-29	-25	-51	-50	-37	24%	24%	24%
Groyne extended 6 m past existing	-56	-30	-21	-42	-47	-32	49%	49%	49%

- *Note: O'Connell Site 2 shows an increase in near-bank shear stress in the 3D low flows likely due to water level being close to where there is a drop in the bathymetry which also coincides with the gradient in the dx of the model grid. These results have been excluded to avoid skewing results.*

6.10 Summary of finding

Some of the key finding from this study are summarised below:

- The 2D simulations generally showed slightly greater median reduction in bed shear stress, due to pile fields, compared with the 3D simulations (i.e. comparing no pile field and pile field scenarios). The 2D and 3D models use different bed shear stress equations and different input variable, therefore the modelled shear stresses are not comparable. However, the similarity of the median percentage reductions provides confidence in the use of 2D models as a design tool.

- The design variations which resulted in the greatest reduction in percent change in shear stress (relative to the existing design) include:

    - Increasing the exposed pile height above the riverbed,
    - Decreasing groyne porosity, and
    - Staggering the piles (i.e. each pile field comprising two stagger rows).

- Varying the groyne angle from 45^{o} to 90^{o} (relative to upstream flow) had minimal influence on shear stress reduction (relative to variation in other pile field design parameters). However, reducing the groyne angle relative to upstream flow (i.e. 45^{o} to upstream flow) increases the groyne length for a given top- and toe-of-bank alignment. This requires additional piles and is therefore associated with a higher design cost.

- Critical shear stress of the channel boundary material, and shear stress and velocity reductions are all important design variables to determine the likelihood of scour and sediment deposition surrounding pile fields. However greater research is required to provide confidence in channel boundary shear stress thresholds for 2D model outputs.

- The shear stress reductions determined in this study are significantly less than those estimate in the shear stress reduction approach as outlined in the Technical Guidelines for Waterway Management (DSE, 2007). Results from this study were used to develop a revised shear stress reduction approach (Section 7.4).

7 Toward the development of hydraulic guidelines

7.1 Review of current approaches

There are three common approaches to pile field design used within Australia:

1. the notional line of attack approach,
2. the shear stress reduction approach, and
3. 2D hydraulic modelling

These three approaches were summarised in Section 3. The application of the notional line of attack and the shear stress reduction approaches require significant engineering judgement based on an understanding of river hydraulics and fluvial geomorphology. There are several issues with both approaches which are summarised in Table 21. No design guidelines were identified for 2D modelling based pile field design approach.

The notional line of attack approach has been used for decades however there is limited theoretical basis for its application. The approach leads to the design of pile fields which are angled relative to flow direction. Decreasing the angle relative to direction of upstream flow (i.e. from 90º to 45º) leads to increased groyne length for a given top- and toe-of-bank alignment. There is limited theoretical justification for significantly angled pile fields based on the scientific literature (Dyer, 1995) and results in this study. This study found minimal impact of the groyne angle in either the 2D or 3D modelling assessments. There is some anecdotal evidence that increasing the groyne angle with the angle of flow assists in shedding flood debris from the groynes which can create blockages and increases turbulent flow. The shedding of debris has potential to reduce such blockages.

The shear stress reduction approach was developed based on a straight flume where flow velocity is predominately in one direction. River hydraulics, especially within meandering reaches, are far more complex with three-dimensional flow patterns which can impact on near-bank velocity profiles and associated shear stress. As a result, the shear stress reductions estimated by the approach are likely overpredictions as has been shown in this study. Furthermore, the approaches do not differentiate reductions for different flow heights. The modelling undertaken in this study indicates the impacts on shear stress significantly reduces with flow height over the elevation of the piles.

The aim of the shear stress reduction approach is to reduce shear stress to a level that enabled deposition of entrained bed sediments. This was typically achieved by determining a target shear stress less than the critical shear stress of bed sediments. However, this study has identified that the shear reductions are unlikely to be less than that of typical bed sediment's critical shear stress (i.e. sands to gravels). However deposition surrounding piles can still occur as a result of localised reductions in sediment transport capacity (due to the shear stress reduction) and the supply of sediment from the upstream reach.

The following section outlines recommendations and considerations required in the design of pile fields groynes for bank stabilisation.

Table 21. Issues associated with pile field design approaches.

Approach	Major issues
The notional line of attack approach	• No theoretical basis • Leads to angled pile fields which increase groyne length and number of piles in each groyne
The shear stress reduction approach	• Assumes shear stress reduction similar for all flow events • Overpredicts shear stress reduction • Developed in straight flume so likely over predicting shear reductions • Does not consider reductions up a sloped embankment

7.2 Design approach

These design guidelines are not intended as a prescriptive set of procedures, rather key recommendations, and factors for designers to consider. Designing river restoration works requires detailed understanding of site characteristics including hydraulics, sediment transport processes, bed and bank substrate, and riparain and watering ecology (including presence/absence of wood borers). As a result, designers need significant expertise beyond the advice that can be provided within this guideline document.

The approached used for the design of pile fields depends on the level of risk at the site and/or with the proposed works. For example, the value of the project (i.e. construction cost), value of assets threatened and the complexity of channel erosion/deposition processes can all impact the level of risk associated with a design. A range of design approaches with varying degrees of rigour are outlined in .

Table 22. Design approach recommended based on the level of risk

Design approach	***Suitability***
Engineering judgement (suitable experience and geomorphic understanding) with limited modelling or quantitative geomorphic analysis. Design may be informed by the Notional Lines of Attack approach (likely results in a conversative design).	Suitable for **low risk** sites, where the cost of works are less than \$50,000 and erosion processes are relatively well understood and no major assets are threatened.
Engineering judgement coupled with quantitative geomorphic analysis including hydraulic modelling of existing conditions to inform the revised shear stress reduction approach as outlined in Section 4 (i.e. no detailed modelling of design scenarios).	Suitable for **moderate risk** sites, where the cost of works are between \$50,000 - \$500,000 and no major asset is under threat.
Engineering judgement coupled with quantitative geomorphic analysis and detailed 2D or 3D modelling of differing design arrangements to optimise the design.	Suitable for **high risk** sites, where the cost of works is greater than \$500,000 and/or a major asset is under threat. Also, may be appropriate where there are very high velocity and shear stress values across the site which increases the erosion risk. For example, if shear stress across the site is greater than 100 N/m² for the existing condition than modelling of different configuration may be required to determine if a suitable reduction can be achieved.

7.3 Probability and risk of failure

The reduction in shear stress and/or velocity predicted from either detailed modelling, or the revised shear stress reduction approach (shear stress only) can be used by designers to determine the probability of failure over the vegetation establishment phase and beyond. To determine the probability of failure the designer will need to know:

1. The probability of alternate flow events and flow rates occurring
2. The shear stress and/or velocity associated with those flow events
3. The temporal variation in critical shear stress and/or non-scour velocity of the bank material (i.e. rate of decline in erosion matting, and timeframes for the establishment of ground covers and structurally diverse riparian vegetation).

A detailed understanding of these metrics can be used by river engineers and river managers predict whether or not a project is likely to be successful (i.e. whether vegetation is likely to reach a level of maturity such that it can provide the long term erosion protection) before the occurrence of a flood event that has the potential to erode the material on which the vegetation is being established. During the vegetation establishment phase, the resistance of that vegetation increases and as a consequence becomes more resilient to flood events. However, with the passage of time, there is increasing likelihood of a large flood event that has the capacity to erode the bed and bank material. The designer can use these metrics to identify the probability of vegetation being eroded during the critical vegetation establishment period, prior to the decay of the pile fields. The designer will need to identify an appropriate probability of success commensurate with the consequence failure and an acceptable level of risk, over the vegetation establishment phase, for the works and site. The level of shear stress reduction afforded by the pile fields , and hence the porosity, height, and spacing of pile fields, should be adjusted such that the probability of failure, creates an acceptable risk of failure.

7.4 Revised shear stress reduction approach

The existing shear stress reduction approach (DSE, 2007) provides designers with an efficient approach to estimate shear stress reduction based on existing hydraulic parameters without undertaking detailed modelling of design scenarios. However, this study has identified the shear stress reductions estimated from the approach are greater than that estimated via the modelling undertaken for this investigation. As a result, relying on the existing shear stress reduction approach could give designers an unrealistic prediction of shear stress reduction.

A revised shear stress reduction approach has been developed based on the modelling outputs of this study. This revised approach allows designers to estimate shear stress reductions without undertaking detailed modelling of design scenarios. This helps reduce design costs for low-risk projects.

This revised shear stress reduction approach is considered an interim design approach until more comprehensive scenario testing can be undertaken for a range of pile field configurations and flows (see recommendations in Section 8). However, this revised approach can provide designers with a greater level of confidence in shear stress reduction estimates.

The relationship between distance downstream from the first pile field and median shear stress reduction within upstream embayment for the 3D and 2D simulations (low flow, mid flow and high flow) are shown in Figure 54 to Figure 59. These plots can help designers determine the likely shear stress reductions based on typical pile field arrangements. The suggested bank shear stress reductions based on the flow height (relative to exposed pile height) are summarised in Table 23. The reduction estimates were developed from typical pile field spacings which are shown in Table 23. The results are only valid for the pile field spacing provided in Table 23. More rigorous modelling of scenarios is required to provide designers with suitable reductions for alternative spacing scenarios. Alternatively, designers can model alternative spacing scenario to determine their own site-specific shear stress reductions.

Table 23. The revised shear stress reduction approach

Pile field spacings from first pile field	Shear stress reduction based on flow height relative to exposed pile height (H)		
	1H	2H	3-4H
7-10 m, 7-10 m, 10-15 m, 10-15 m, 10-15 m, 15-20 m, 15-20 m, 15-20 m etc.	50%	40%	30%

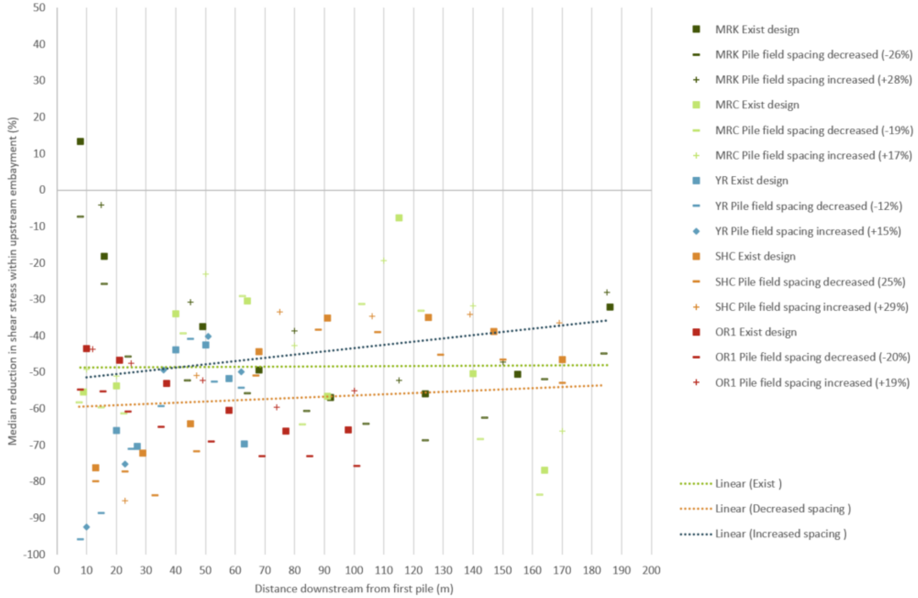

Figure 54. *Relationship between distance downstream from first pile field and median shear stress reduction within upstream embayment (3D low flow scenario - flow height 1.5 m above riverbed). Showing trendline for existing design spacing, decreased spacing and increased spacing scenarios. Note: O'Connell Site 2 shows an increase in near-bank shear stress in the 3D low flows likely due to water level being close to where there is a drop in the bathymetry. Therefore, these results have been excluded.*

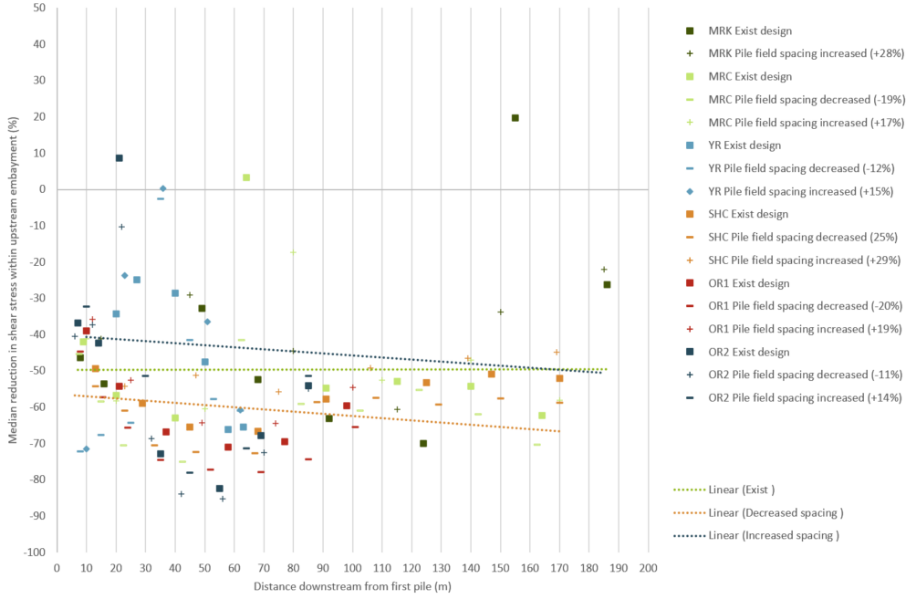

Figure 55. *Relationship between distance downstream from first pile field and median shear stress reduction within upstream embayment (2D low flow scenario - flow height 1.5 m above riverbed). Showing trendline for existing design spacing, decreased spacing and increased spacing scenarios. Note: There is a significant increase in near-bank shear stress in one embayment at the Mary River Kenilworth site in the 2D low flows. These results have been excluded to avoid skewing results.*

Figure 56. *Relationship between distance downstream from first pile field and median shear stress reduction within upstream embayment (3D mid flow scenario - flow height 3 – 3.5 m above riverbed). Showing trendline for existing design spacing, decreased spacing and increased spacing scenarios.*

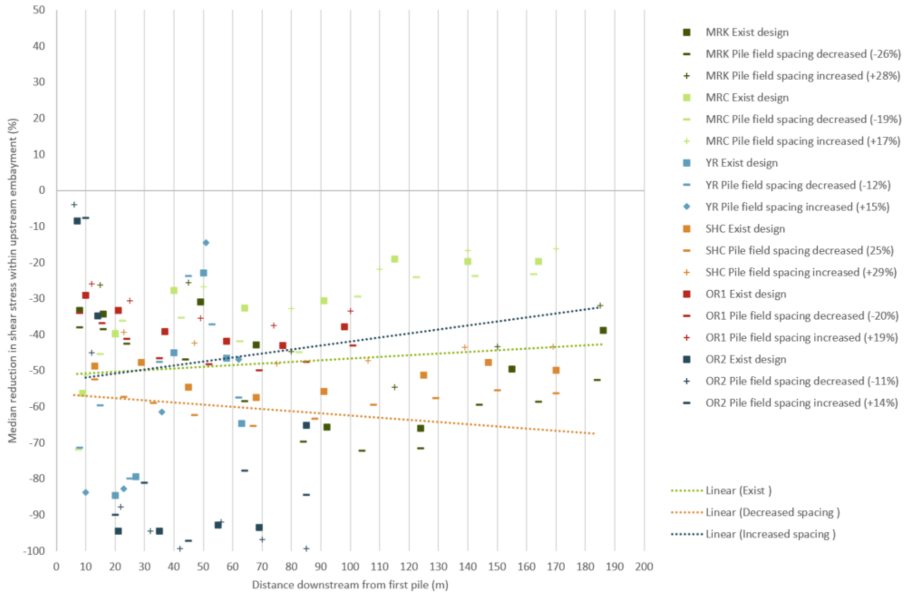

Figure 57. *Relationship between distance downstream from first pile field and median shear stress reduction within upstream embayment (2D mid flow scenario - flow height 3 – 3.5 m above riverbed). Showing trendline for existing design spacing, decreased spacing and increased spacing scenarios.*

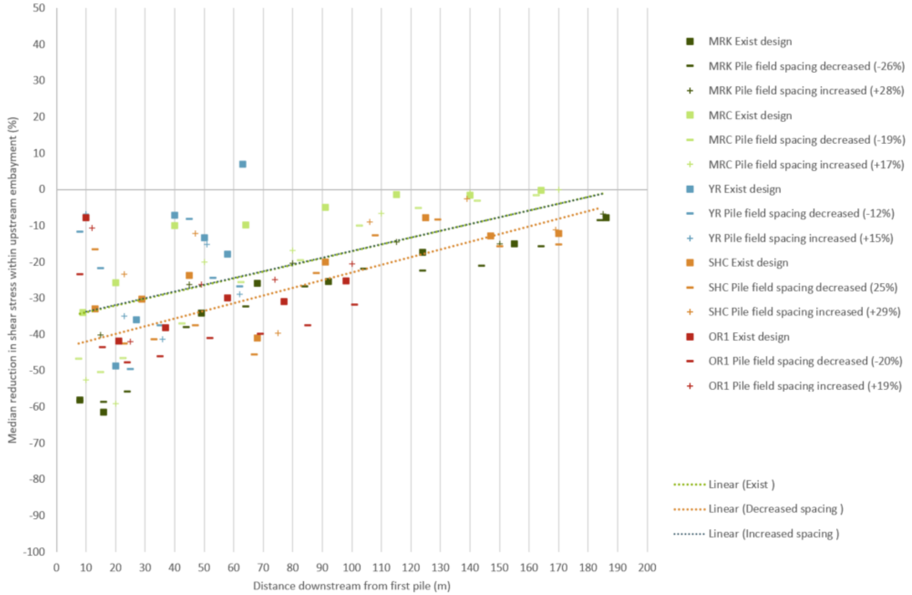

Figure 58. *Relationship between distance downstream from first pile field and median shear stress reduction within upstream embayment (3D high flow scenario – flow overtopping pile fields). Showing trendline for existing design spacing, decreased spacing and increased spacing scenarios.*

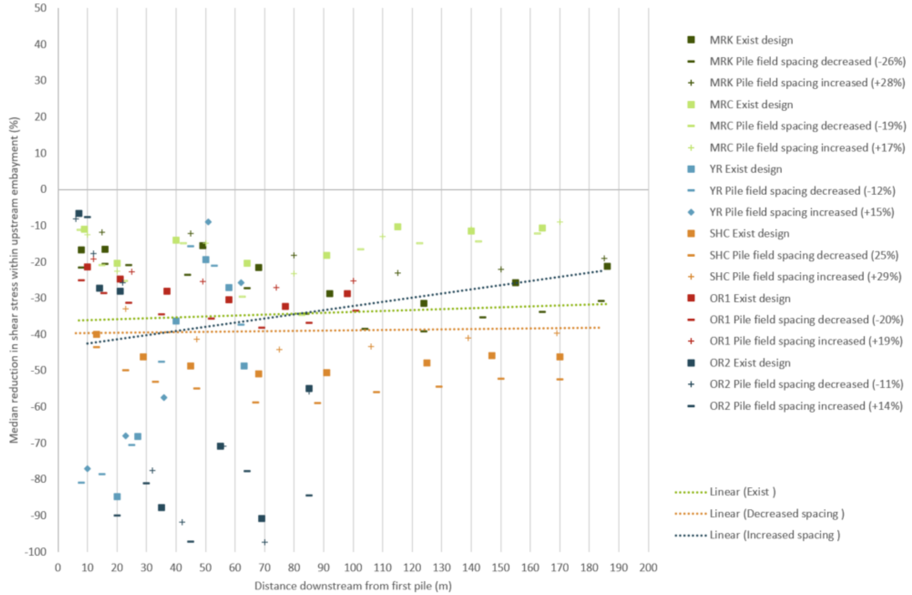

Figure 59. *Relationship between distance downstream from first pile field and median shear stress reduction within upstream embayment (2D high flow scenario – flow overtopping pile fields). Showing trendline for existing design spacing, decreased spacing and increased spacing scenarios.*

7.5 Design considerations

Positioning

The positioning of the groynes (i.e. from the most upstream location and to the most downstream location) is a critical decisions in the design process. One of the most common failure mechanisms for groynes is outflanking. Outflanking occurs when bank retreat and associated bend migration upstream of the groynes results in erosion of the bank surrounding the structural works.

Positioning of the groynes requires a detailed understanding of the bank substrate, erosion processes and bend migration trajectory. Many bank erosion sites are the result of a down valley meander migration process (Figure 60). As a result, retreat rates at the upstream end of the site wane over time until they are negligible. In some instances, deposition may occur as part of a concave bench formation process. As a result, position of groynes is crucial for project success. If groynes are not positioned far enough upstream the works can be outflanked. Conversely if groynes are positioned too far upstream it may result in a significantly more expensive project, as structural works are installed in areas where they're not required.

The use of multi-temporal spatial analysis should be used to assess bend migration trajectory and help the positioning of groynes.

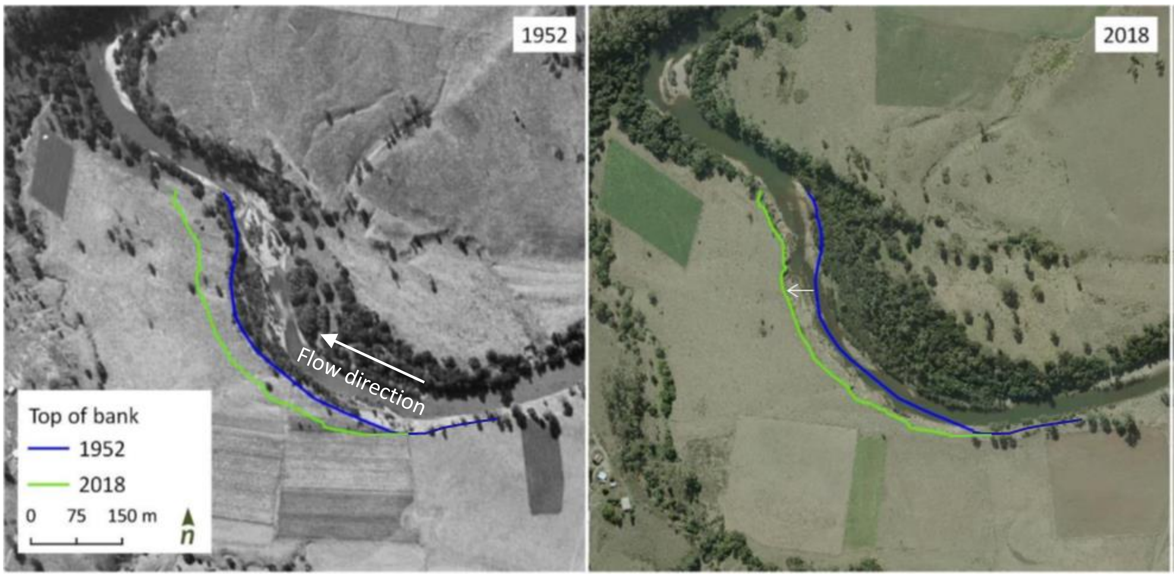

Figure 60. *Example of a down valley meander migration processes*.

Design toe of bank

The positioning of pile fields requires a design toe of bank (Figure 61). The riverward side of the groynes then extend out to the design toe of bank. The zone of hydraulic influence (i.e. where there is significant reduction in velocity and shear stress) is controlled by the location of the design toe of bank (see Figure 61). If the design toe of bank is extended out from the existing toe of bank a more significant zone of hydraulic influence is created.

Modelling as part of this study indicated that extending the design toe of bank out from bank area has variable impact on the near bank shear stress. In some case extending the groyne length past toe of bank results in a marginally increase in near bank shear stress (this is likely due to an increase in the groyne cross-sectional area relative the total channel cross-sectional area). However, extending the design toe of bank out in the riverward direction can have a range of unintended consequences including:

- Redirection of flow towards the centre of the river can reduce the flow sinuosity and increase velocity. This process could result in increased flow energy being exerted on adjacent bank areas and is analogous to 'river straightening'.
- Create excess scour on the riverward side of groynes by extending the groynes into the higher flow velocity areas of the channel.
- Impacts on instream habitat values such as changes to benthic habitat and infilling of pools.

As a result, the positioning of the design toe of bank needs to balance shear stress reductions with these broader geomorphic and river health implications. Where bank reprofiling is proposed as part of the stabilisation strategy the design toe of bank should extent at least 1 m from the toe of the bank batter to limit the risk of toe scour.

Figure 61. *Example pile field design showing the design toe of bank set out from toe of the bank and the zone of hydraulic influence.*

Spacing
Initial spacing of piles should be based on the revised shear stress reduction approach as outlined in Section 7.2. This approach is considered appropriate for low or moderate risk as outlined above in .

At high-risk sites more rigorous modelling and analysis of the pile field spacing should be undertaken using a 2D model. This study found that reducing spacing by approximately 25% (relative to design based on the Technical Guidelines shear stress approach) resulted in an additional shear stress reduction ranging approximately 5 – 10%. Reduction in shear stress, due to decreased pile field spacing, was generally most notable in the low flow scenarios.

Groyne angle
This study identified increasing or decreasing groyne angle relative to upstream flow has marginal impact on near-bank shear stress. As a result, a more cost effective design may be achieved by increasing the groyne angle to flow. However, the designer will need to consider other factors including bend curvature and debris shedding processes as part of the design.

Height of piles
The exposed pile height was identified to have a significant impact on shear stress reduction across the bank area. However, the exposed height is limited by the embedment depth of the pile. An approach for determining pile diameter and depth of embedment based on the estimated drag, impact and resistant forces

applied to a pile within a waterway is outline in the Technical Guideline for Waterway Management (Attachment C). As a result, the exposed height of the pile will be limited by:

- The length and diameter of available piles
- The depth to which piles can be driven at the site

For an exposed height of 1.5-2 m with a diameter of 250-350 mm the pile will need to be approximately 6 m in length (depending on the depth of embedment calculations). If higher exposed pile height is required then longer piles are likely to be required such that a suitable embedment depth can be achieved. Increasing pile length will also be accompanied by increasing pile diameter. Both increased length and diameter have the potential to significantly increase the cost to supply and drive piles. As such, increasing pile height may be an effective means of reducing bed shear, but may not be the most cost-efficient approach and alternate approaches such as reduced porosity of pile fields or reduced spacing between pile fields may need to be explored.

No matter the design exposed pile height at the toe of the bank the exposed height should always taper down on the bank side of the works such that the last few piles are no more than 0.5 m above the bank height. This is to reduce the eddying effect on the bank side of the works which could result in scour and potential outflanking. Similarly, the designer should consider complex flow response (i.e. eddying) surrounding pile fields as part of the design process.

Height of pile field on the bank side

The aim of pile fields is to reduce the shear stress across a bank area to a suitable level to increase the likelihood of vegetation establishment. Structurally diverse, remnant standard vegetation will typically be able to resist erosion on many streambank areas (i.e. critical shear stress of approximately 120 N/m^2). However, vegetation takes time to establish and there is a risk high flow events will erode the bank prior to the vegetation reaching maturity.

Velocity and shear stress will reduce along the reprofiled bank (i.e. as flow depth decreases) (Figure 62 to Figure 64). As a result, greater scour protection is required along the lower bank compared to the mid to upper bank area. Over time the boundary resistance of the bank area will also increase as the vegetation establishes (see Figure 67). As a result, the degree of required toe protection will reduce as the vegetation matures.

To determine the optimal height of toe protection up the bank requires a probabilistic cumulative risk of erosion analysis which will require:

- An understanding of the variation of shear stress across the site for different flood magnitudes (see example in Figure 63 and Figure 64). Variation in velocity across the site for different flood magnitudes could also be assessed (Figure 65 and Figure 66).
- An understanding of the temporal variation in bank boundary resistance as the vegetation establishes (i.e. from post construction to full maturity)
- Flood frequency analysis to determine critical scour events for each temporal period

The probabilistic cumulative risk of erosion analysis can help designers determine the optimal height of bank protection works to provide a low likelihood of bank scour which may undermine any revegetation works.

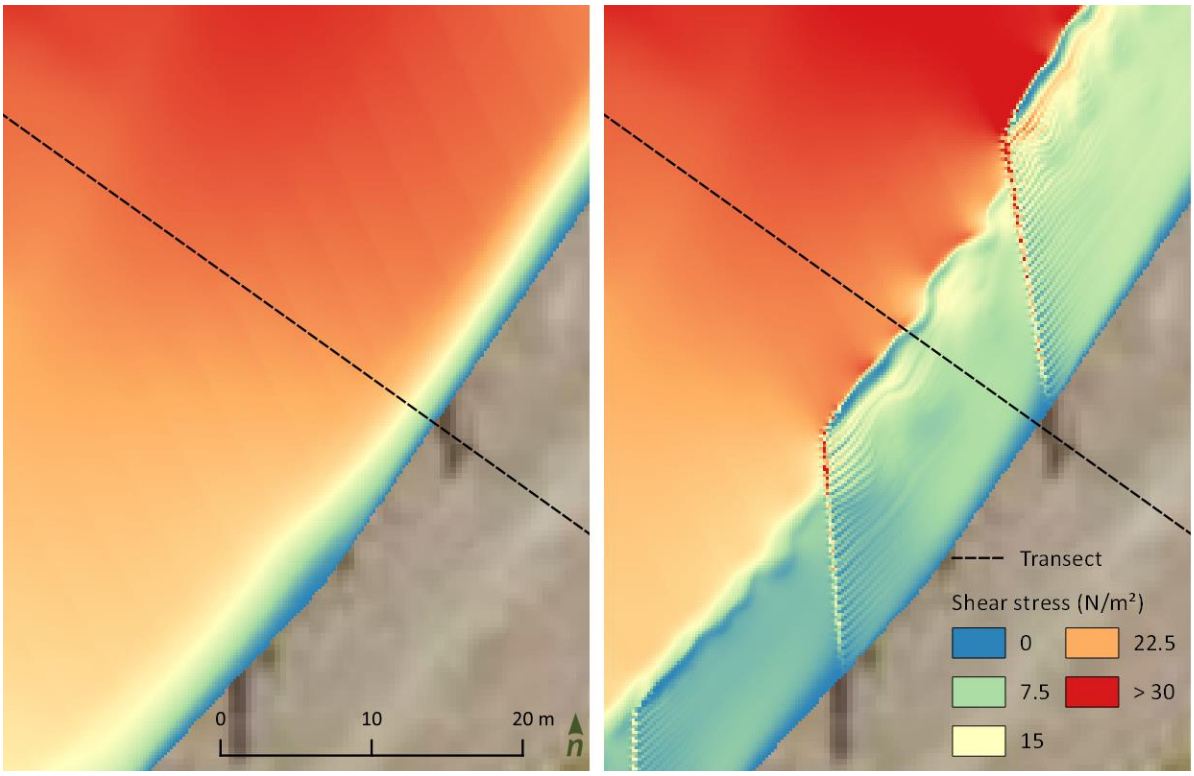

Figure 62. *Distribution of the spatial shear stress (N/m²) at the St Helens Creek site (Low flow, 2D model). No pile field scenario (left), and existing pile field design (right).*

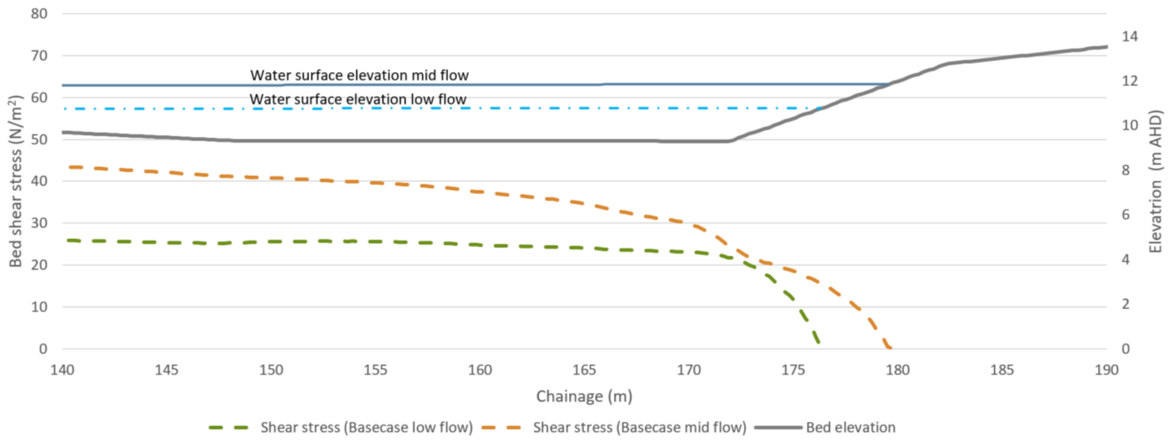

Figure 63. *Cross-section showing decrease in shear stress across the bank face as flow depth decreases (no pile field scenario)*

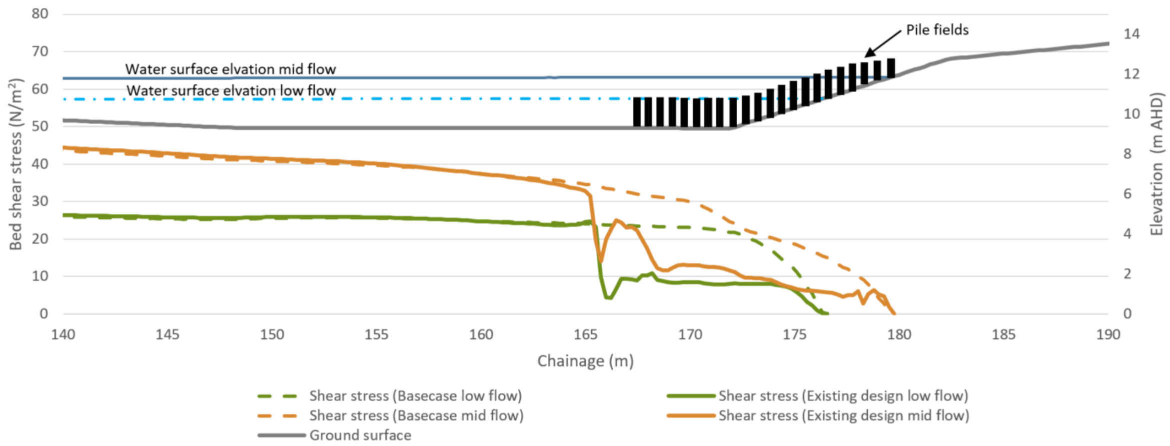

Figure 64. *Cross-section showing reduction in shear stress across the bank face due to pile fields (existing pile field scenario). Note: both the no pile field and pile field scenarios incorporated a reprofiled bank (to an approximate gradient of 1V:3H). Therefore, the shear stress reduction due to pile field bank stabilisation (and associated bank reprofiling) compared to eroding bank conditions (i.e. vertical banks) is likely greater than results indicate.*

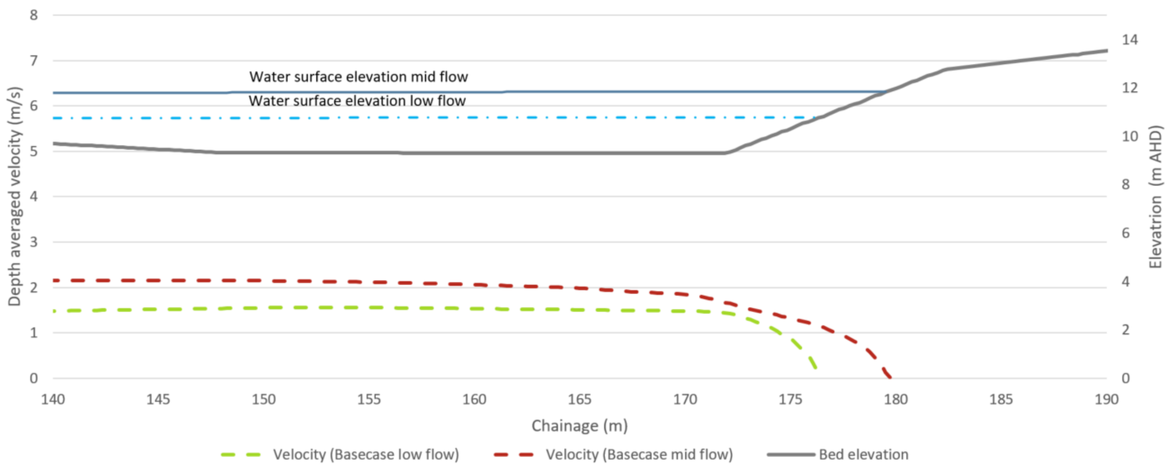

Figure 65. *Cross-section showing decrease in velocity across the bank face as flow depth decreases (no pile field scenario)*

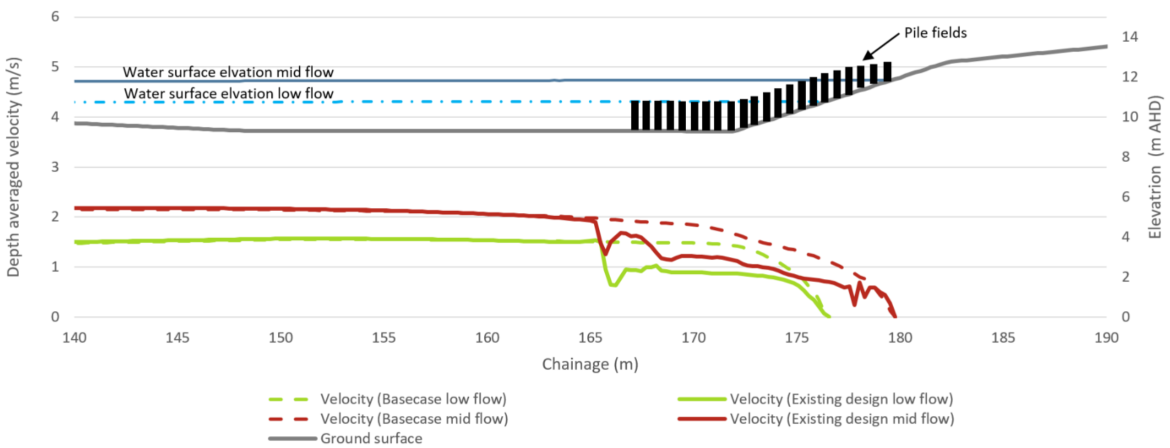

Figure 66. *Cross-section showing reduction in velocity across the bank face due to pile fields (existing pile field scenario).*

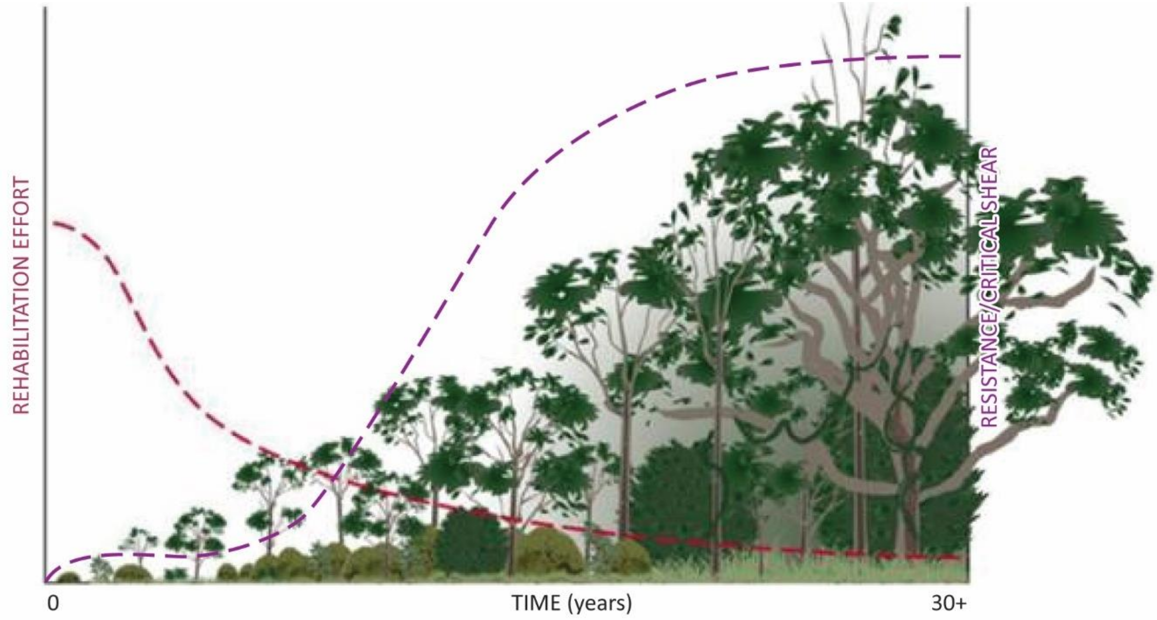

Figure 67. *Progressive long-term improvement in river health and erosion resistance with gradual reduction in rehabilitation effort (source: Department of Sustainability and Environment (2004)).*

8 Summary and recommendations

Pile field groynes are an effective tool for reducing the rates of bank erosion and allowing riparian vegetation establishment (Figure 68). Pile fields in conjunction with appropriate riparian management practices can significantly help reduce fine sediment loads in Great Barrier Reef catchments.

Figure 68. *Vegetation establishment at a site over time following bank reprofiling and pile field installation, the photo on the left is 2017 and the photo on the right is less than five years later.*

This study has identified several improved design approaches which can significantly increase both the cost effectiveness and the certainty around design performance. Some of the major outcomes of the project include:

- Confidence that 2D modelling provide a comparable reduction in shear stress compared to 3D modelling.
- The groyne angle has minimal impact on shear stress reduction, future designs can increase the angle relative to upstream flow and result in reduced pile field length and cost.
- The revised shear stress approach developed in Section 4 provides designers with greater confidence in the actual shear stress reductions for a variety of flow heights.

However, the study has also identified several future research opportunities that can further improve the understanding of the hydraulic performance of pile field groynes. These include:

1. More detailed investigation into shear stress calculations in 2D models and how these compare to values determined from one-dimensional, reach averaged, model outputs.
2. Many published shear stress thresholds were developed from one-dimensional, reach averaged, model outputs. As a result, many of these values (and potentially non-scour velocity values) may need to be updated based on 2D model outputs.
3. Further expansion of the revised shear stress reduction approach as outlined in Section 7.2 (i.e. application at additional sites).
4. Investigate the impacts of pile fields on near-bank velocity. There is less uncertainty surrounding the calculation of velocity in hydraulic models compared to shear stress.
5. Undertake field monitoring of pile field sites to assess velocity profiles (and/or shear stress profiles) to help with the validation of hydraulic models.

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Attachment A
Pile field bank stabilisation specifications

Mary River – Kenilworth Park
The bank stabilisation works for the Mary River Kenilworth site includes bank reprofiling, pile fields and vegetation establishment. The functional design to stabilise the eroding bank is presented in Figure A1. The components of the stabilisation works include:

- Reprofiling the bank to a gradient of 1V:3H, to provide a suitable environment for vegetation establishment
- Installation of pile fields at the toe of bank
- Establishment of riparian vegetation along the reprofiled bank and overbank zone
- Installation of rock revetment at the upstream end of the works

The recommended stabilisation works extended across a batter length of 220 m, and comprised eight piles fields (approximately 308 individual piles). Specifications for the bank reprofiling and pile fields are shown in Table A1 and A2.

During site set-out an additional pile field was added at the downstream end of the site. The post-construction works are shown in Figure A2.

Table A1. Bank reprofiling specification table – Mary River Kenilworth Park

Material specifications	Units	Value
Total length of works	m	220
Maximum fill and batter slope	m	1V:2.5H
Estimated volume of cut	m^3	1800
Estimated volume of fill	m^3	1800

Table A2. Pile field specification table – Mary River Kenilworth Park

Pile field number	Pile field length (m)	Pile field tail length (m)	Porosity (%)*	Number of 6 m piles	Number of piles split piles (4m & 2m)	Total number of piles	Spacing between pile fields (m)
P1	22	2	50	44	4	48	30
P2	20	2	50	40	4	44	30
P3	17	2	50	34	4	38	30
P4	16	2	50	32	4	36	15
P5	14	2	50	28	4	32	15
P6	18	2	50	34	6	40	8
P7	14	2	50	26	6	32	8
P8	17	2	50	32	6	38	
			TOTAL	**270**	**38**	**308**	

*pile diameter and clear space between piles = 250 mm

Figure A1. Functional design of stabilisation works – Mary River Kenilworth Park (Alluvium, 2014)

Figure A2. Pile field groynes - Mary River (Kenilworth Park)

Mary River – Carters Site

The bank stabilisation works for the Mary River Carters site includes bank reprofiling, pile fields and vegetation establishment. The functional design to stabilise the eroding bank is presented in Figure A3. The components of the stabilisation works include:

- Reprofiling the bank to a gradient of 1V:3H, to provide a suitable environment for vegetation establishment
- Installation of pile fields at the toe of bank
- Establishment of riparian vegetation along the reprofiled bank and overbank zone
- Installation of rock revetment at the upstream end of the works

Specifications for the bank reprofiling and pile fields at Mary River Carters site are shown in Table A3 and A4 respectively.

During site set-out the location and orientation of pile field 4 and 5 was adjusted, and an additional pile field was added at the downstream end of the site. The stabilisation works include nine pile fields over a batter length of 235 m. The post-construction works are shown in *Figure* .

Table A3. Bank reprofiling specifications table – Mary River Carters

Material specifications	Units	Value
Length of batter slope	m	235
Bank batter slope	m/m	1V:3H
Estimated volume of cut material	m^3	12,100
Estimated volume of fill material	m^3	1,100

Table A4. Pile field specifications table – Mary River Carters

Pile field number	Pile field length (m)	Pile field tail length (m)	Individual pile length (m)	Pile diameter (mm)	Clear space between piles (mm)	Number of piles
P1	21	2	6	300	300	39
P2	21	2	6	300	300	39
P3	17	2	6	300	300	32
P4	12	2	6	300	300	24
P5	11	2	6	300	300	22
P6	18	2	6	300	300	34
P7	23	2	6	300	300	43
P8	22	2	6	300	300	41
					TOTAL	**274**

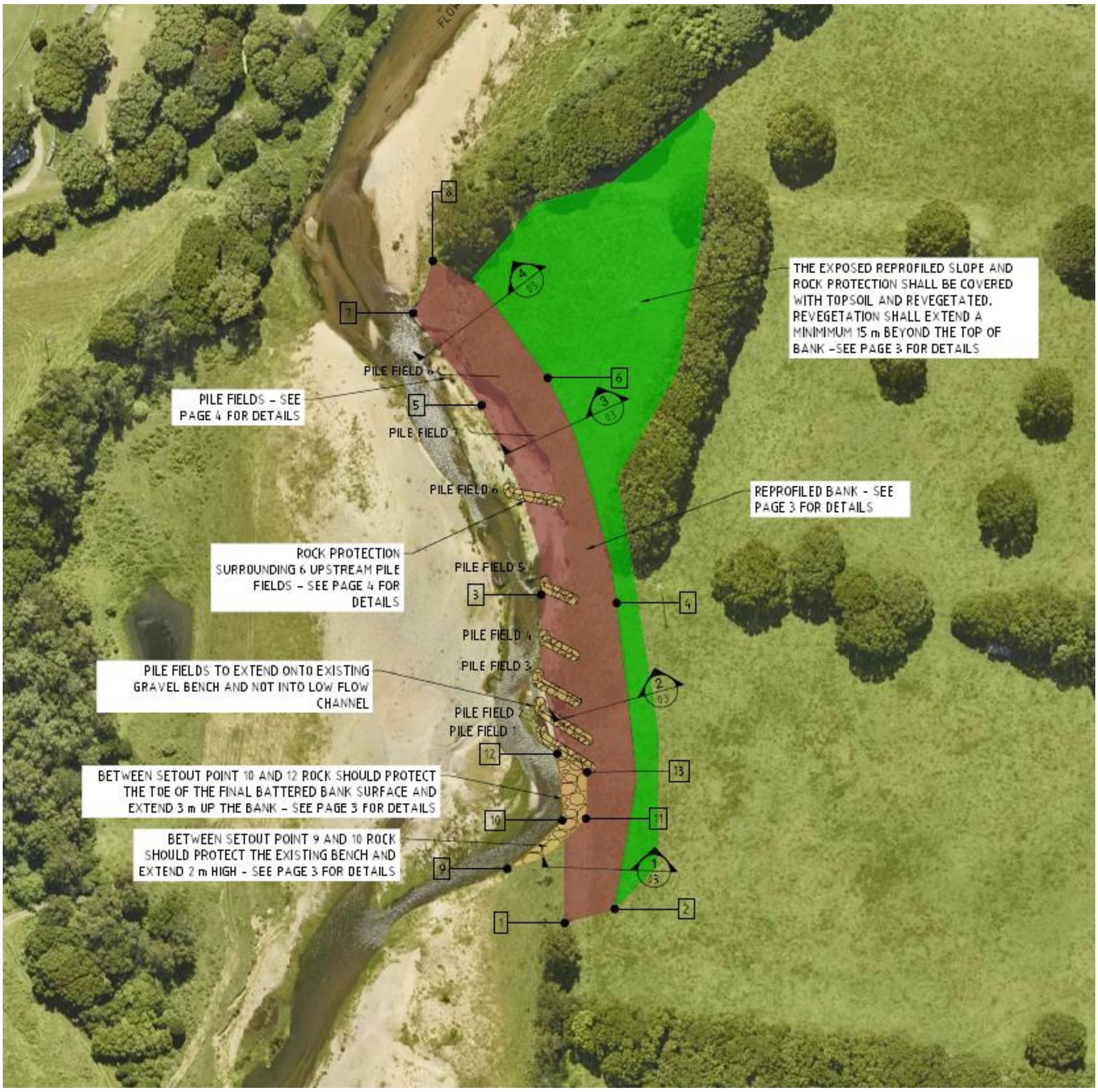

Figure A3. Functional design of stabilisation works – Mary River Carters site (Alluvium, 2019)

Figure A4. Pile field groynes - Mary River Carters Site

O'Connell River Site 1

The bank stabilisation works for the O'Connell River Site 1 include revegetation on a battered stream bank, supported by structural pile fields and associated rock beaching bank protection. The functional design to stabilise the eroding bank is presented in Figure A5. The components of the stabilisation works include:

- Reprofiling the bank to a gradient of 1V:3H, to provide a suitable environment for vegetation establishment
- Installation of pile fields along the lower bank
- Establishment of riparian vegetation along the reprofiled bank and floodplain zone
- Installation of rock revetment at the upstream end of the works to protect the toe of the bank and reduce the risk of outflanking

The stabilisation works extend across a batter length of 140 m, and comprise of seven fields (approximately 294 individual piles). Specifications for the bank reprofiling and pile fields are shown in Table A5 and A6 respectively. The post-construction works are shown in *Figure* .

Table A5. Bank reprofiling specification table – O'Connell River Site 1

Material specifications	Units	Value
Total length of works	m	140
Maximum fill and batter slope	m	1V:3H
Estimated volume of cut from upper bank	m^3	1500
Estimated volume of fill to be placed between piles	m^3	1500

Table A6. Pile field specification table – O'Connell River Site 1

Field no.	Field length (m)	Tail length (m)	Porosity (%)	Total no. Of 6m piles	Total no. Of piles to be split (4m & 2m)	Total no. Of piles	Field spacing (m)
P1	17	2	50	34	4	38	8
P2	17	2	50	34	4	38	8
P3	17	2	50	34	4	38	8
P4	21	2	50	42	4	46	15
P5	21	2	50	42	4	46	15
P6	20	2	50	40	4	44	15
P7	20	2	50	40	4	44	
			TOTAL	**266**	**28**	**294**	

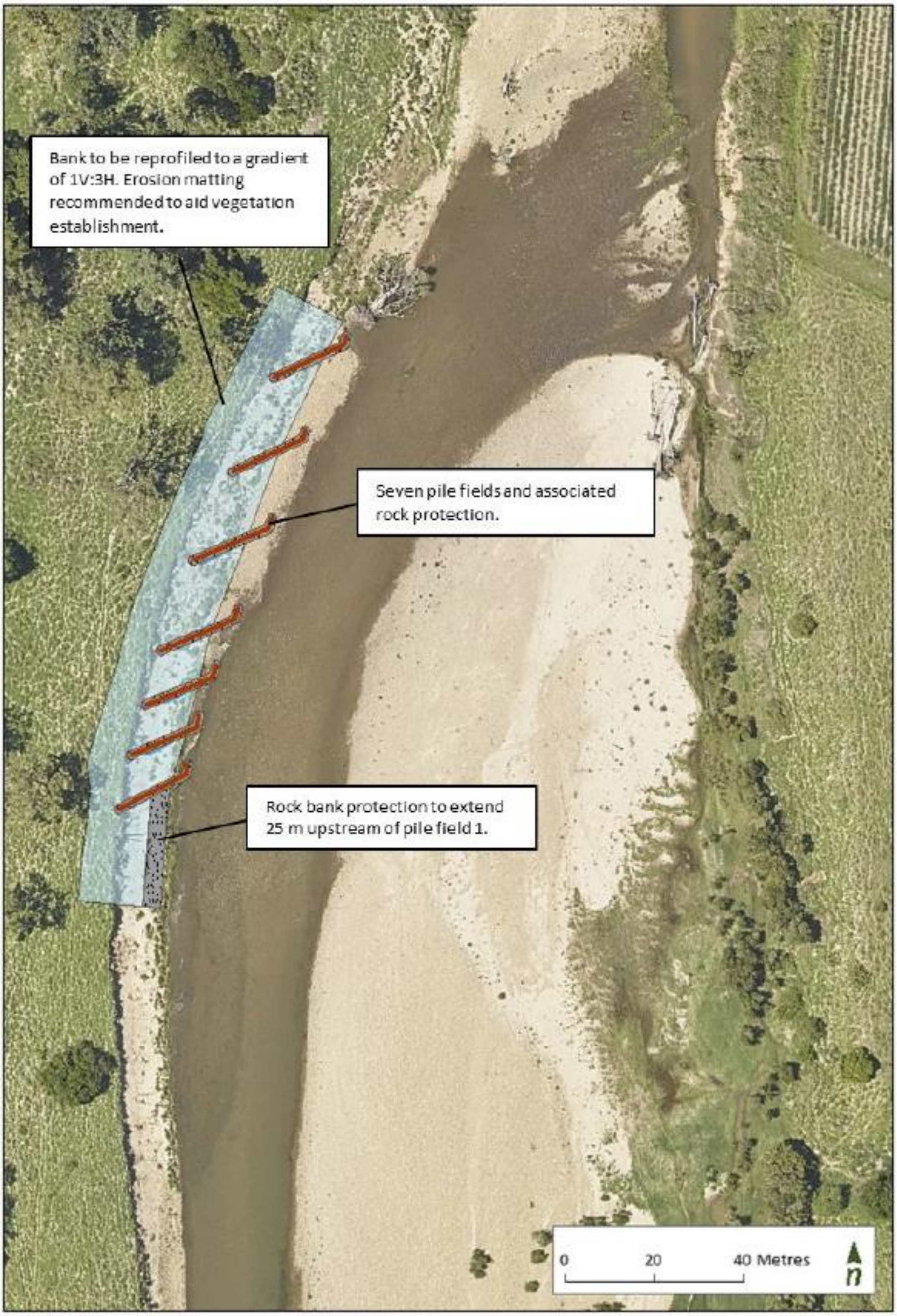

Figure A5. Functional design of stabilisation works – O'Connell River Site 1 (Alluvium, 2015)

Figure A6. Pile field design – O'Connell River Site 1

O'Connell River Site 2

The bank stabilisation works for the O'Connell River Site 2 include revegetation on a battered stream bank, supported by structural pile fields and associated rock beaching bank protection. The functional design to stabilise the eroding bank is presented in Figure A7. The components of the stabilisation works include:

- Reprofiling the bank to a gradient of 1V:3H, to provide a suitable environment for vegetation establishment
- Installation of pile fields along the lower bank
- Establishment of riparian vegetation along the reprofiled bank and floodplain zone
- Installation of rock revetment at the upstream end of the works to protect the toe of the bank and reduce the risk of outflanking

The recommended stabilisation works extend across a batter length of 105 m, and comprise of seven fields (approximately 244 individual piles). Specifications for the bank reprofiling and pile fields are shown in Table A7 and Table A8 respectively.

During site set-out an additional pile field was added at the downstream end of the site. The post-construction works are shown in Figure A8.

Table A7. Bank reprofiling specification table – O'Connell River Site 2

Material specifications	Units	Value
Total length of works	m	105
Maximum fill and batter slope	m	1V:3H
Estimated volume of cut material	m^3	2500
Estimated volume of fill material	m^3	500

Table A8. Pile field specification table – O'Connell River Site 2

Pile field number	Pile field length (m)	Pile field tail length (m)	Pile diameter (mm)	Clear space between piles (mm)	Total number of piles
P1	14	1.5	250-300	250-300	30
P2	14	1.5	250-300	250-300	31
P3	15	1.5	250-300	250-300	34
P4	16	1.5	250-300	250-300	35
P5	16	1.5	250-300	250-300	36
P6	17	1.5	250-300	250-300	38
P7	18	1.5	250-300	250-300	40
				TOTAL	**244**

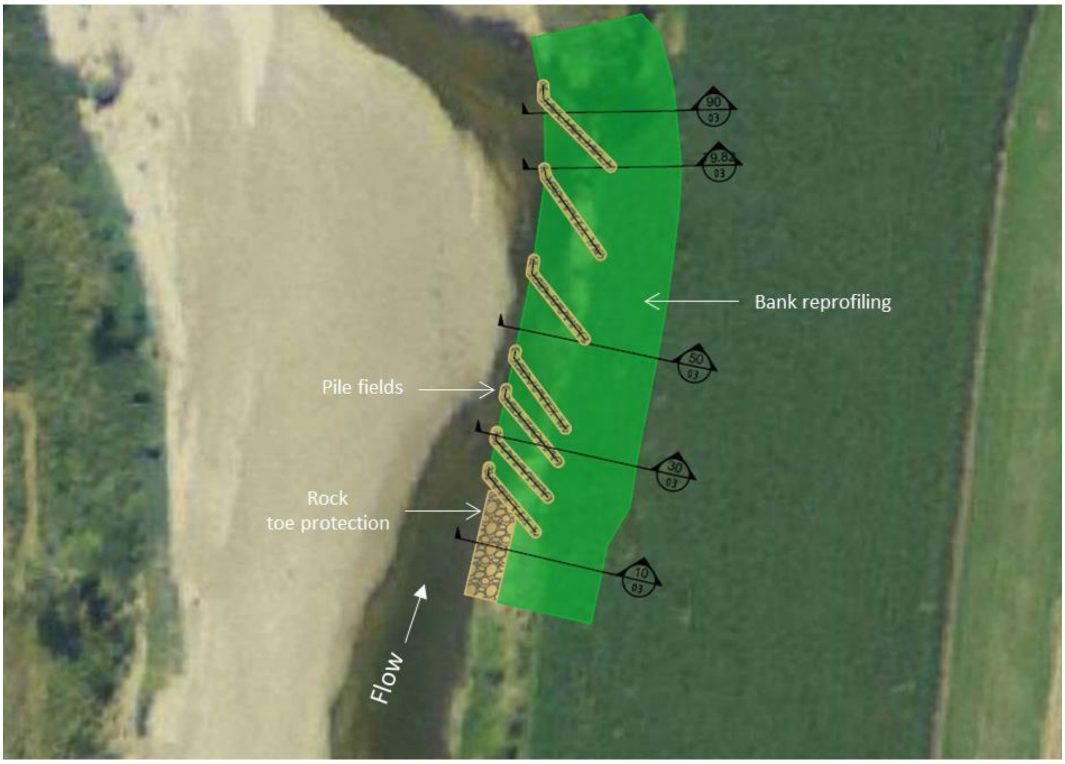

Figure A7. Functional design of stabilisation works – O'Connell River Site 2 (Alluvium, 2018b)

Figure A8. Pile field design – O'Connell River Site 2

St Helens Creek

The bank stabilisation works for the St Helens site include rock toe protection, bank reprofiling, pile fields toe protection and revegetation works. The functional design to stabilise the eroding bank is presented in Figure A 9. The components of the stabilisation works include:

- Reprofiling the bank to a gradient of 1V:3H, to provide a suitable environment for vegetation establishment
- Installation of pile fields along the toe of bank
- Establishment of riparian vegetation along the reprofiled bank and overbank zone
- Installation of rock revetment at the upstream end of the works

The recommended stabilisation works extend across a batter length of 210 m, and comprise of 7 piles fields (approximately 326 individual piles). Specifications for the bank reprofiling and pile fields are shown in Table A9 and Table A10 respectively.

During site set-out an additional two pile fields was added at the downstream end of the site. The post-construction works are shown in Figure A10

Table A9. Bank reprofiling specifications table – St Helens Creek

Material specifications	Units	Value
Length of batter slope	m	210
Bank batter slope	m/m	1V:3H
Estimated volume of cut material	m^3	367
Estimated volume of fill material	m^3	280

Table A10. Pile field specifications table – St Helens Creek

Pile field number	Pile field length (m)	Pile field tail length (m)	Pile diameter (mm)	Clear space between piles (mm)	Total number of piles	Number of piles	
						Individual pile length - 3 m	Individual pile length - 6 m
P1	20	1.5	250-300	250-300	45	-	45
P2	16	1.5	250-300	250-300	37	7	30
P3	17	1.5	250-300	250-300	40	6	33
P4	18	1.5	250-300	250-300	43	7	35
P5	20	1.5	250-300	250-300	46	7	39
P6	24	1.5	250-300	250-300	54	8	46
P7	27	1.5	250-300	250-300	61	7	54
				TOTAL	**326**	**43**	**282**

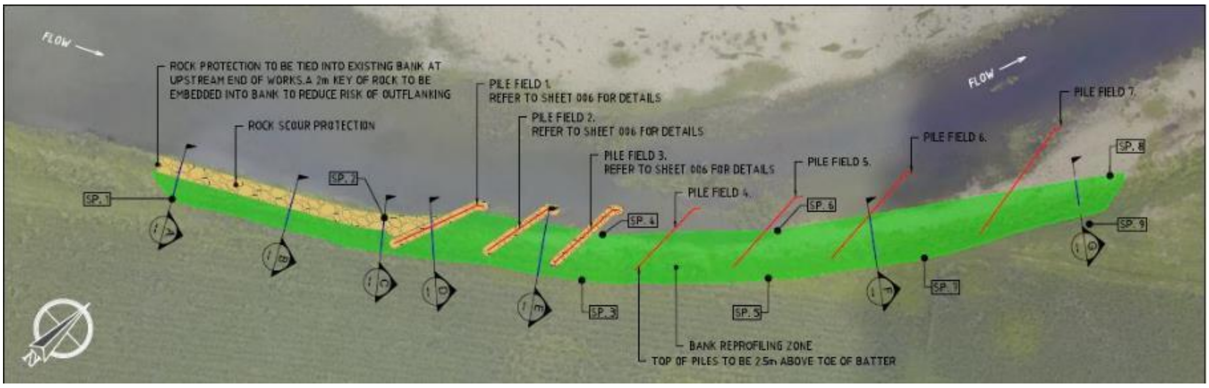

Figure A9. Functional design of stabilisation works – St Helens Creek (Alluvium, 2018c)

Figure A10. Pile field design – St Helens Creek

Molongle Creek

The recommended bank stabilisation works for the Molongle Creek site include bank reprofiling, pile fields toe protection and revegetation works. The functional design to stabilise the eroding bank is presented in Figure A11. The components of the recommended stabilisation works include:

- Reprofiling the bank to a gradient of 1V:3H, at the upstream and downstream extent of the works
- Installation of pile fields along the bank
- Establishment of riparian vegetation along the reprofiled bank and overbank zone

The recommended stabilisation works extend across batter length of 300 m, and comprise of 17 piles fields (approximately 647 individual piles). Specifications for the bank reprofiling and pile fields are shown in Table A11 and Table A12 respectively. The site of the proposed works is shown in Figure A12

Table A11. Bank reprofiling specification table – Molongle Creek

Material specifications	Units	Value
Length of batter slope	m	300
Bank batter slope	m/m	1V:3H
Estimated volume of cut material	m^3	1,508

Table A12. Pile field specifications table – Molongle Creek

Pile field layout	Distance to downstream pile (m)	Length of pile field (m)*	Pile diameter (mm)	Clear space between piles (mm)	Total number of piles (Number)
P1	10	26.0	300	300	43
P2	10	26.0	300	300	43
P3	10	25.5	300	300	43
P4	10	16.1	300	300	27
P5	20	20.9	300	300	35
P6	20	20.8	300	300	35
P7	20	17.6	300	300	29
P8	20	20.9	300	300	35
P9	20	20.2	300	300	34
P10	20	13.9	300	300	23
P11	10	21.2	300	300	35
P12	10	22.4	300	300	37
P13	10	25.3	300	300	42
P14	10	26.9	300	300	45
P15	10	28.1	300	300	47
P16	10	29.6	300	300	49
P17		26.5	300	300	44
				Total	**647**

*Includes 2m tail in downstream direction and 4 piles extended past the crest

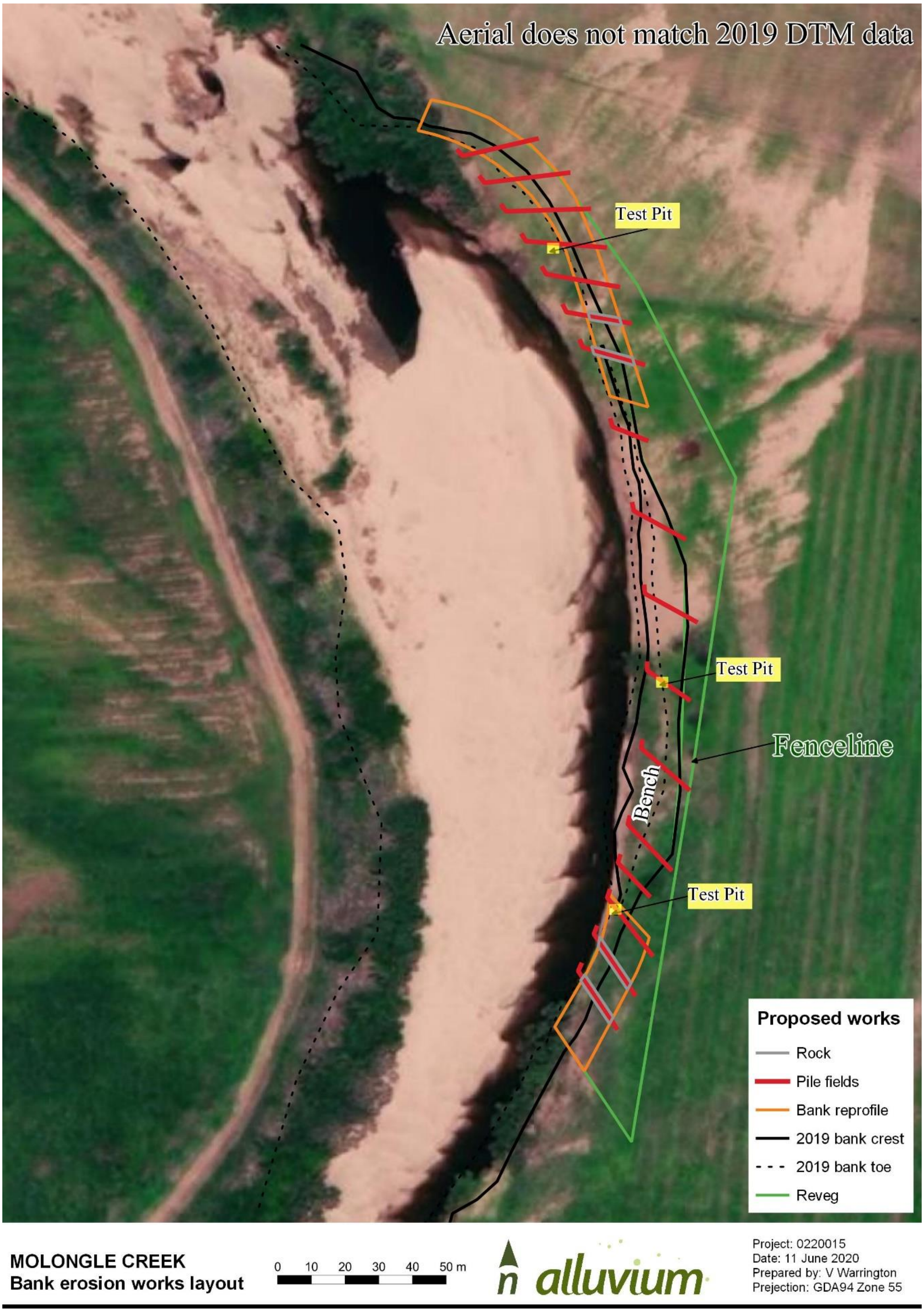

Figure A11. Functional design of stabilisation works – Molongle Creek (Alluvium, 2020)

Figure A12. Molongle Creek Site location

Yarra River

The bank stabilisation works for the Yarra River site include pile fields and revegetation works. The functional design to stabilise the eroding bank is presented in Figure A13. The components of the stabilisation works include:

- Installation of pile fields to realign the toe of bank. This alignment provides a significant depositional zone whilst not confining the main channel adversely.
- Establishment of riparian vegetation along the reprofiled bank and overbank zone

The stabilisation works extend across a stream bank length of 60 m, and comprise of 5 piles fields (approximately 225 individual piles). Specifications for the pile fields are shown in Table A13. The post-construction works are shown in Figure A14.

Table A13. Pile field specifications table – Yarra River

Pile field	Pile field length (m)	Tail length (m)	Total length (m)	Pile diameter (mm)	Porosity	Total number of piles
P1	13	2	15	200	40%	45
P2	13	2	15	200	40%	45
P3	17	2	19	200	40%	57
P4	20	2	22	200	60%	44
P5	15	2	17	200	60%	34
					Total	**225**

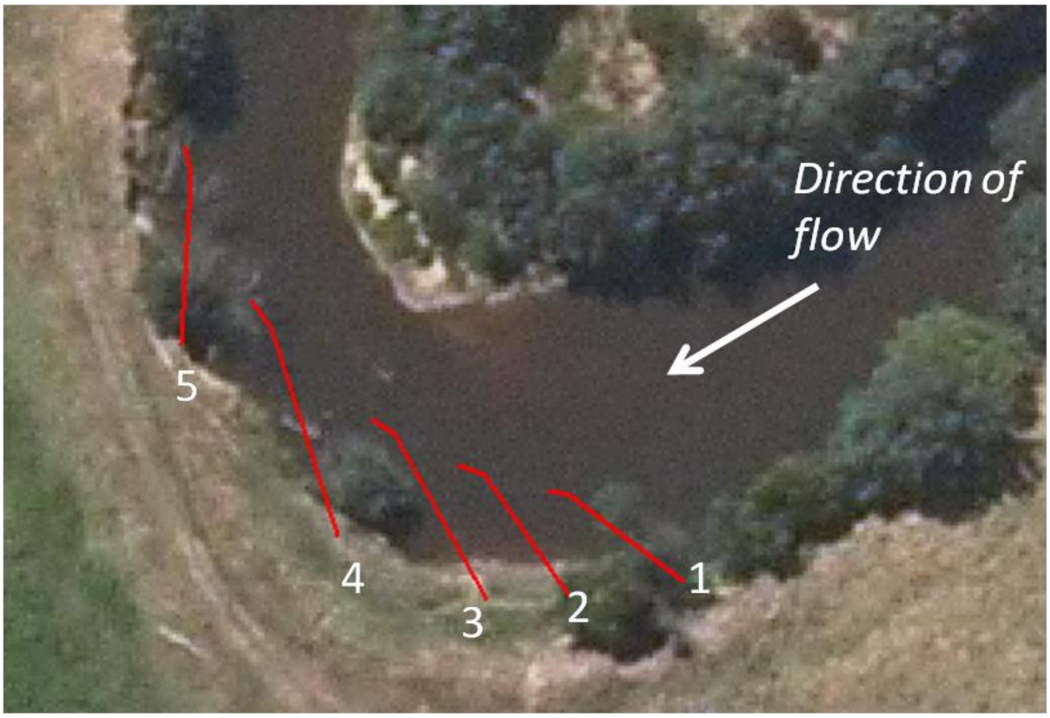

Figure A13. Functional design of stabilisation works – Yarra River (Alluvium, 2008)

Figure A14. Pile field design – Yarra River

Attachment B
3D and 2D bed shear stress results

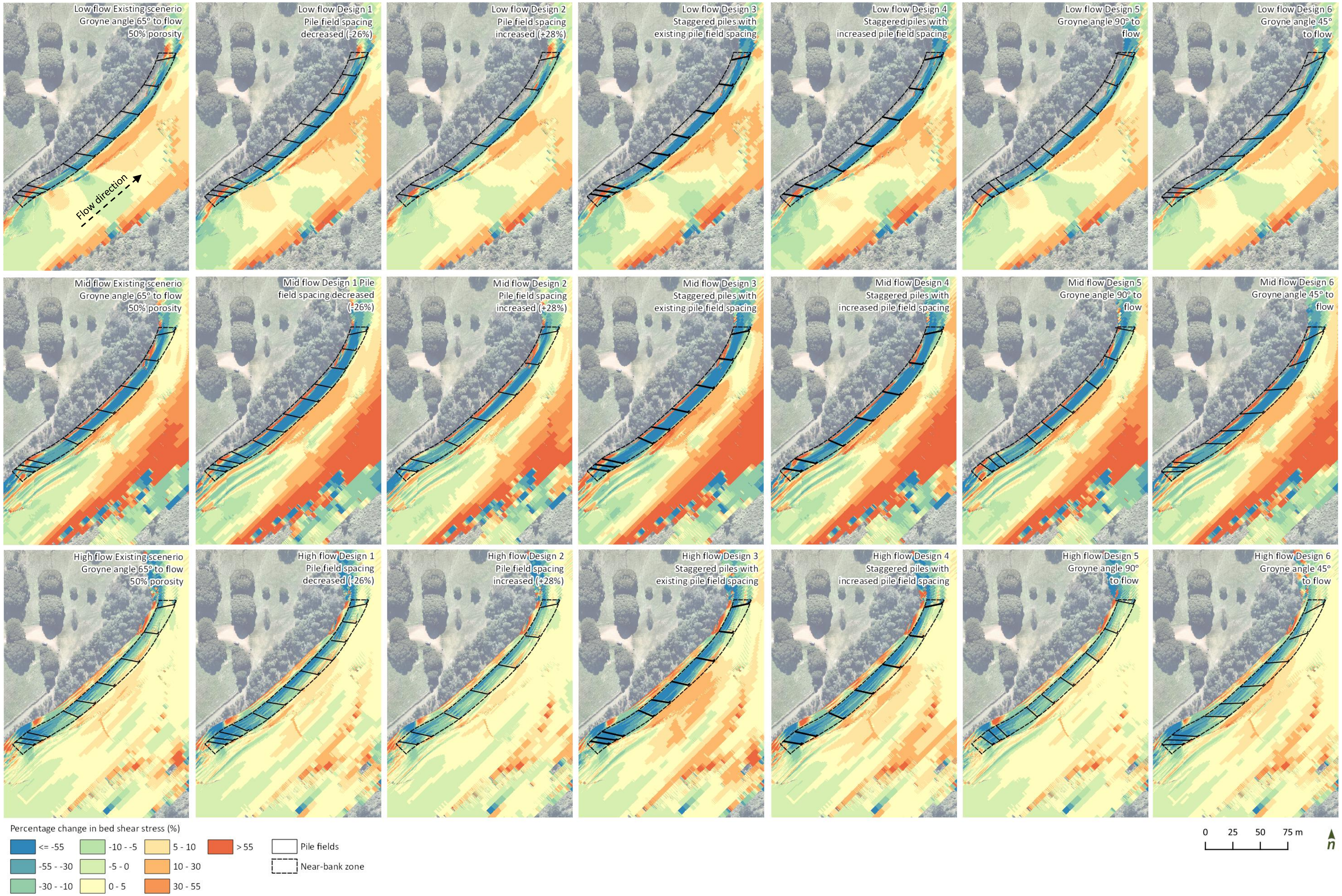

Figure B1. *AEM3D Mary River Kenilworth Site: percentage change in bed shear stress between the various pile field scenarios and no pile field scenario for the low flow (top row), mid flow (middle row), and high flow (bottom row). As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase.*

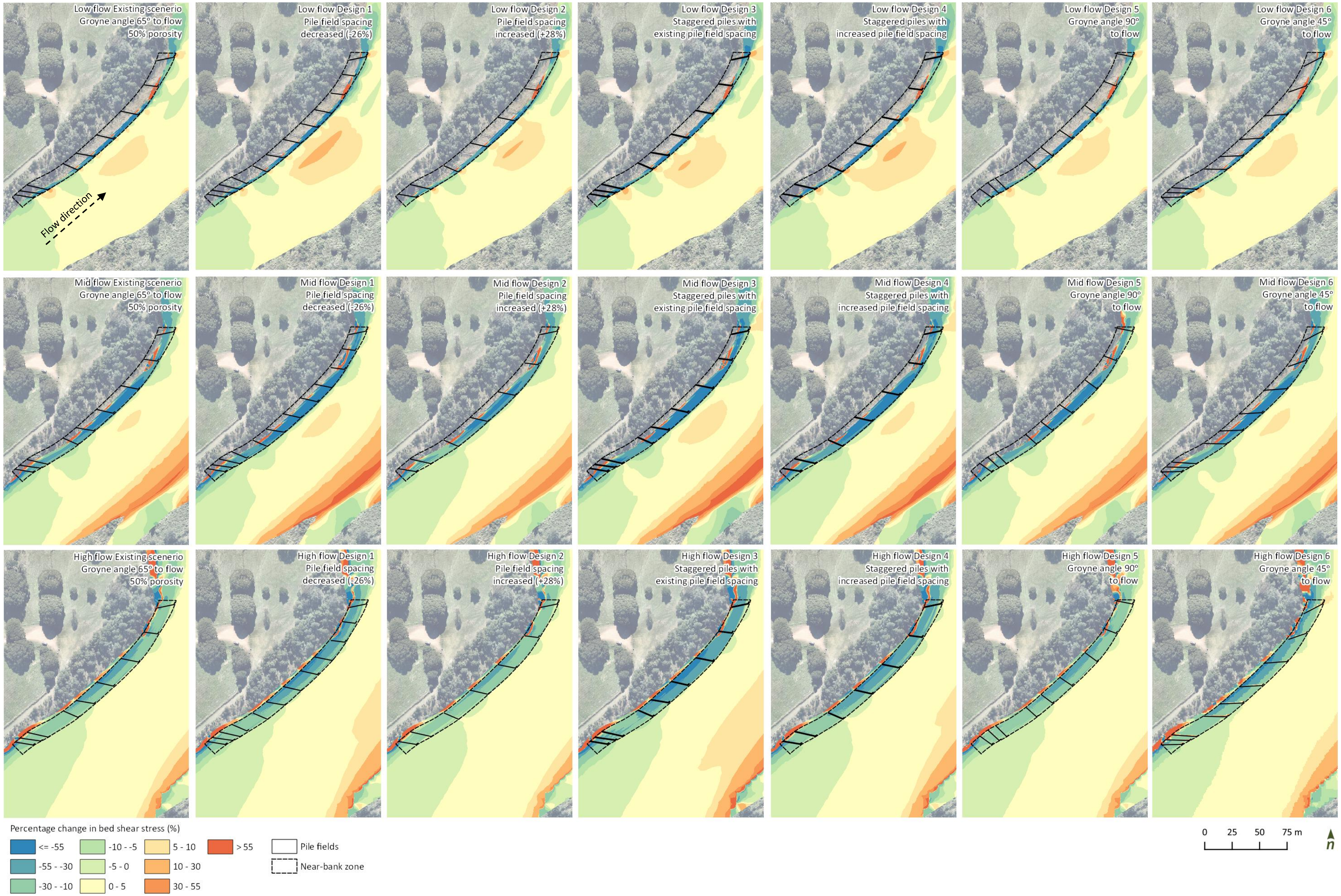

***Figure B2.** HECRAS 2D Mary River Kenilworth Site: percentage change in bed shear stress between the various pile field scenarios and no pile field scenario for the low flow (top row), mid flow (middle row), and high flow (bottom row). As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase.*

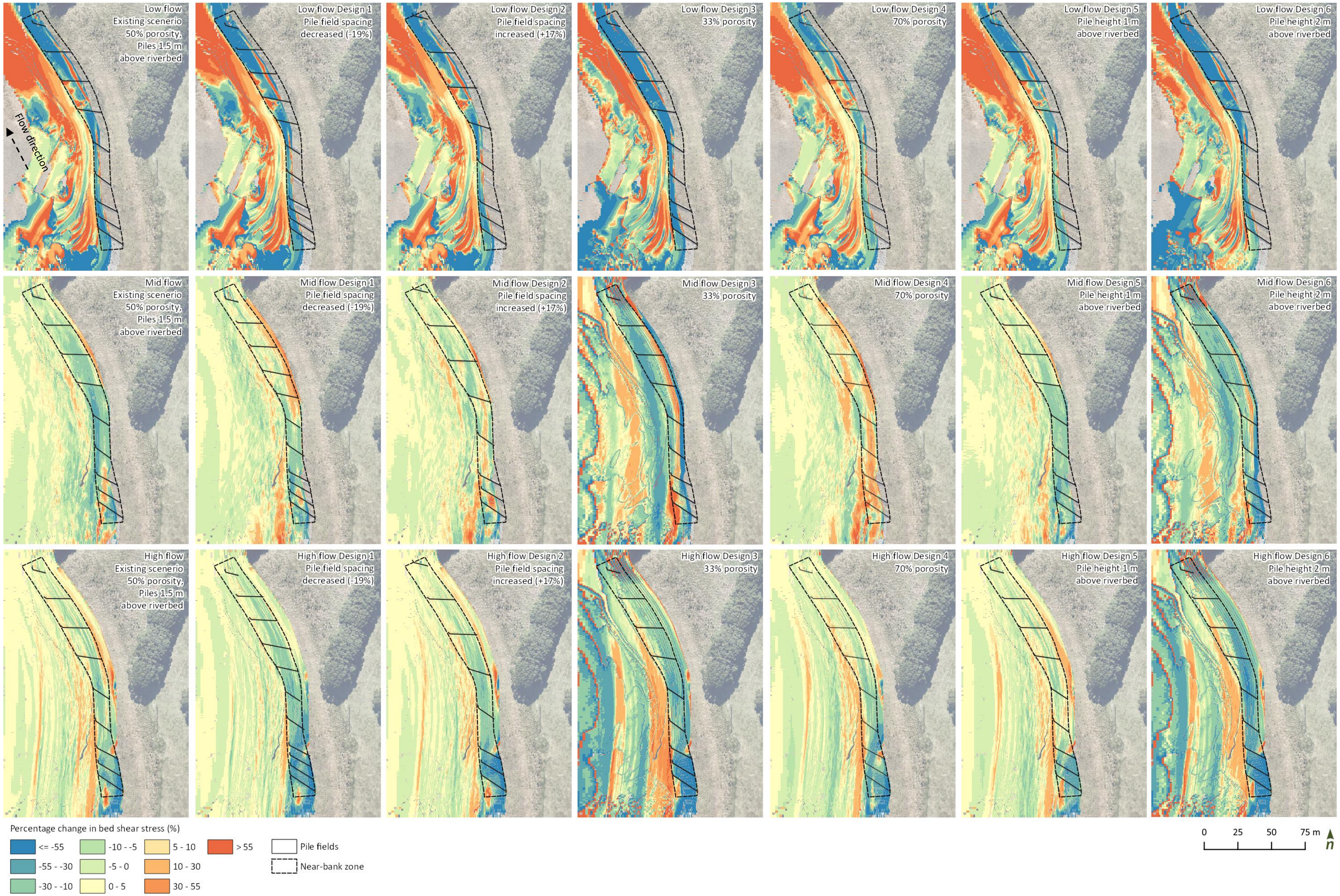

Figure B3. *AEM3D Mary River Carters Site: percentage change in bed shear stress between the various pile field scenarios and no pile field scenario for the low flow (top row), mid flow (middle row), and high flow (bottom row). As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase.*

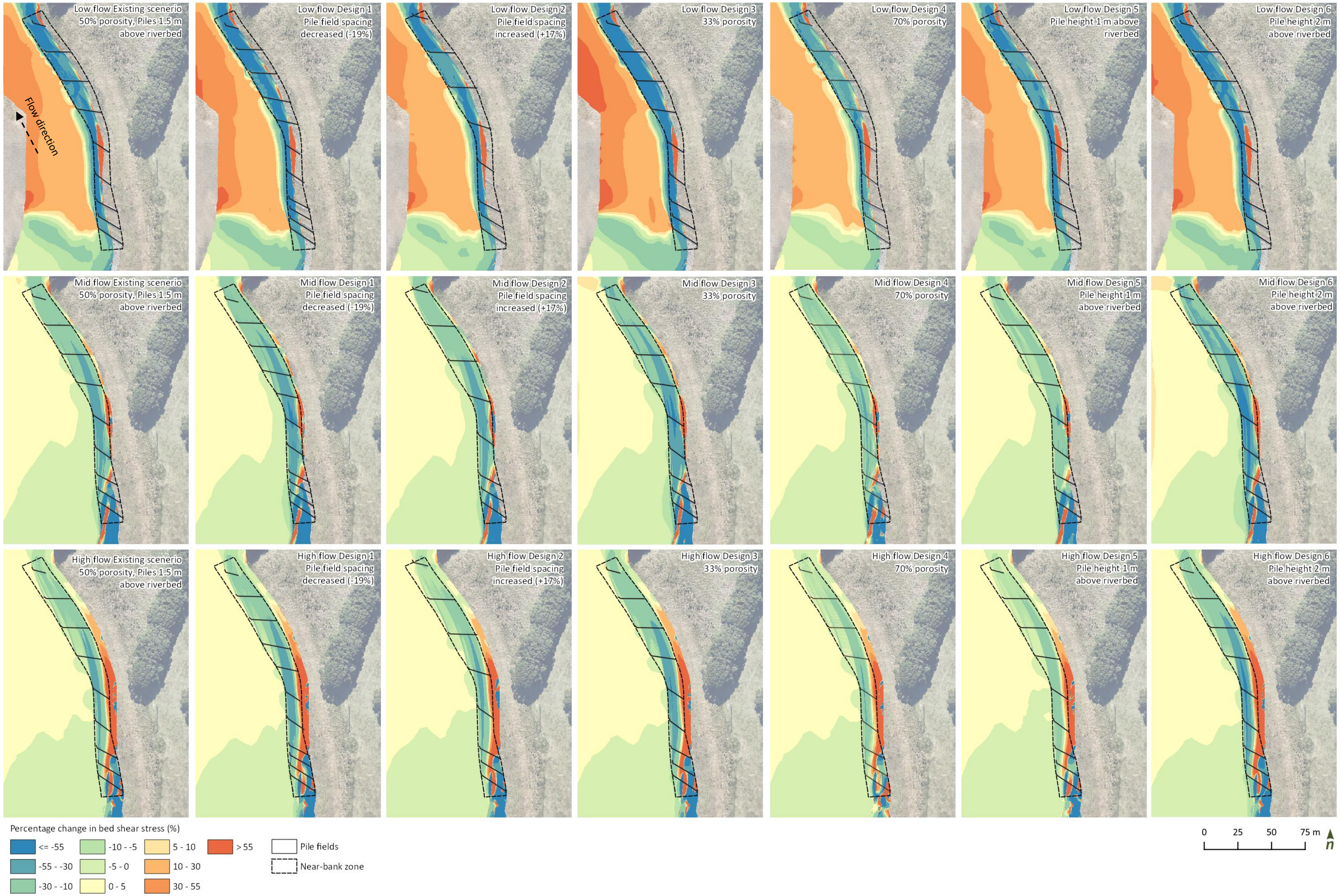

Figure B4. *HECRAS 2D Mary River Carters Site: percentage change in bed shear stress between the various pile field scenarios and no pile field scenario for the low flow (top row), mid flow (middle row), and high flow (bottom row). As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase.*

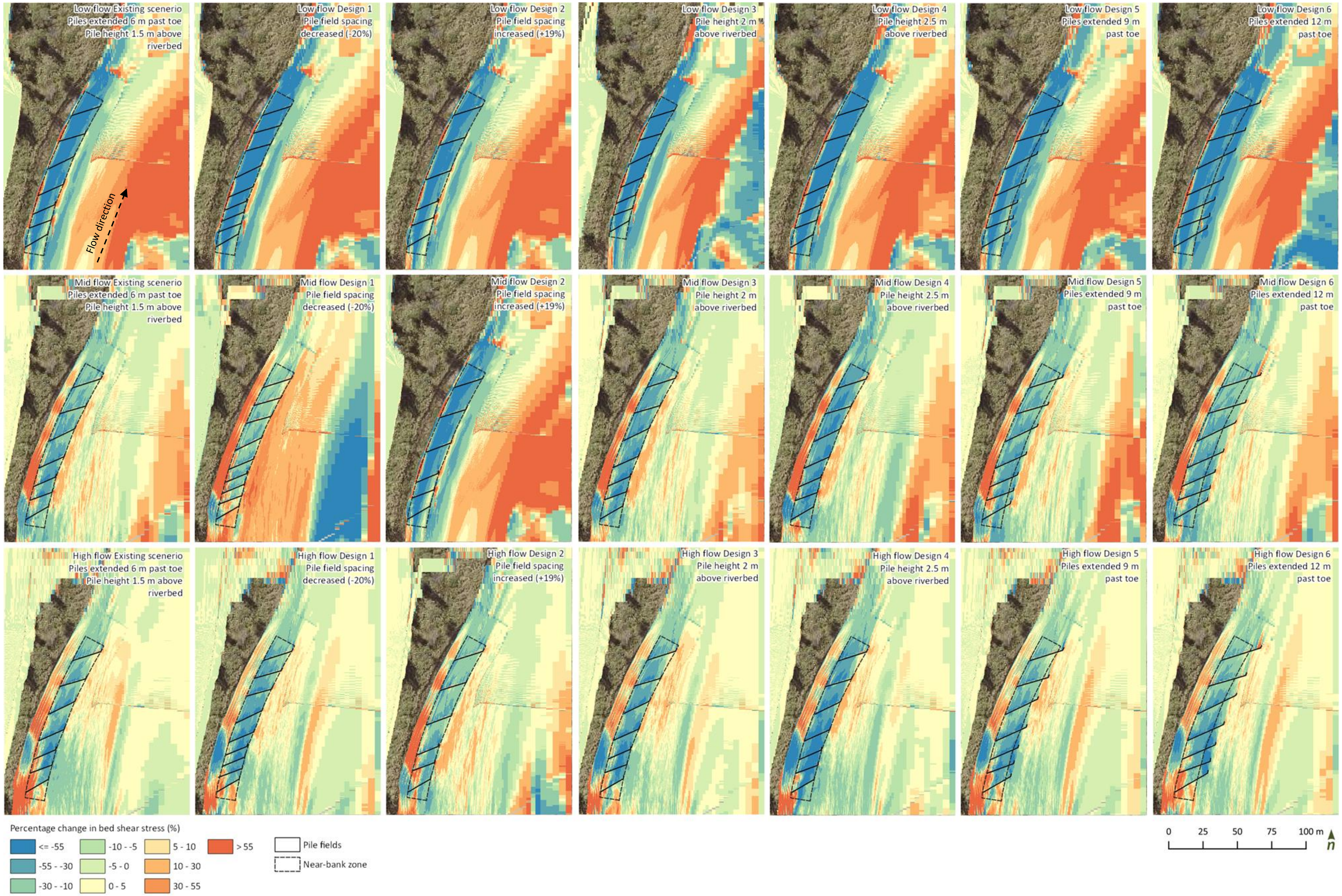

Figure B5. *AEM3D O'Connell River Site 1: percentage change in bed shear stress between the various pile field scenarios and no pile field scenario for the low flow (top row), mid flow (middle row), and high flow (bottom row). As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase.*

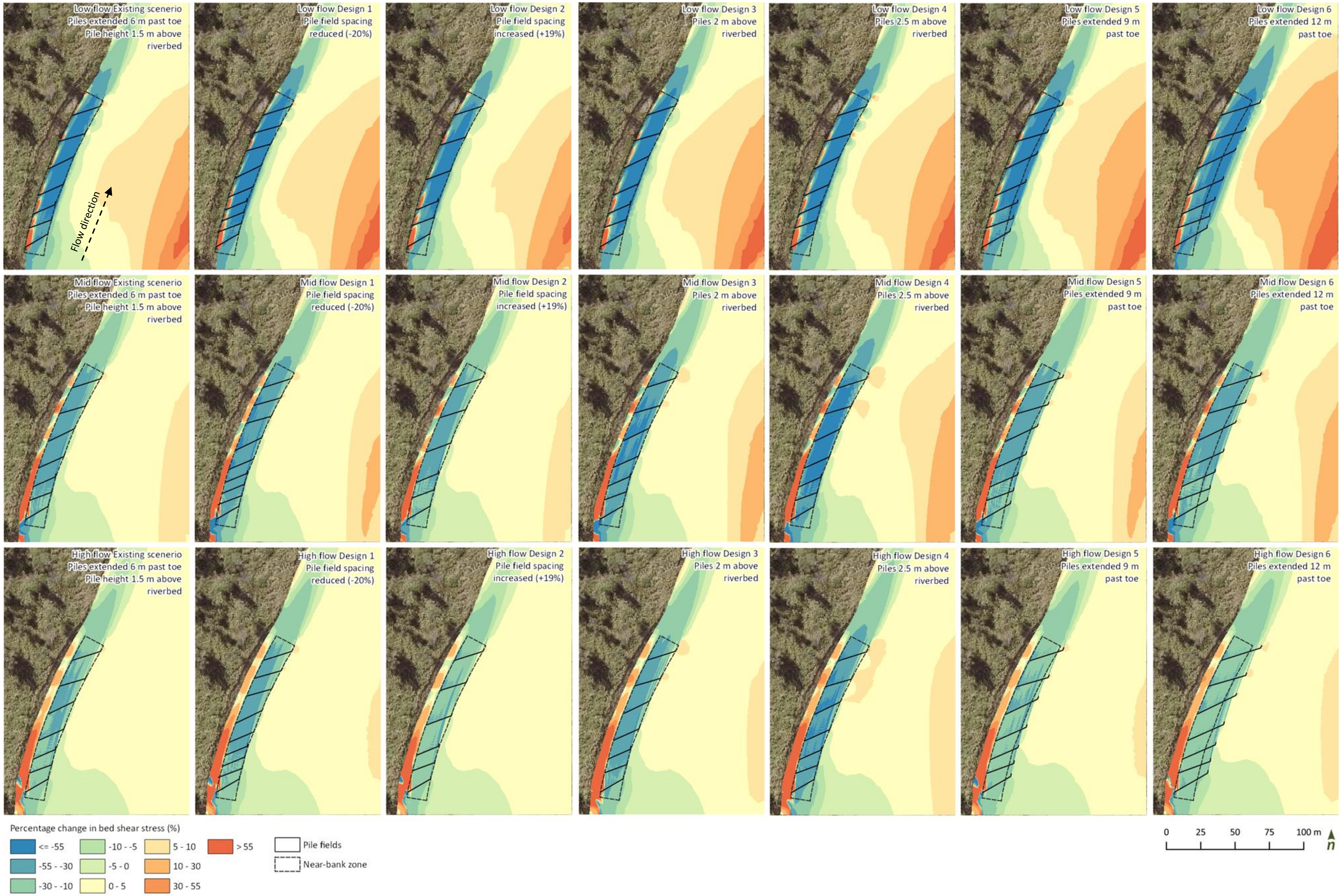

Figure B6. *HECRAS 2D O'Connell River Site 1: percentage change in bed shear stress between the various pile field scenarios and no pile field scenario for the low flow (top row), mid flow (middle row), and high flow (bottom row). As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase. – fix figure text!*

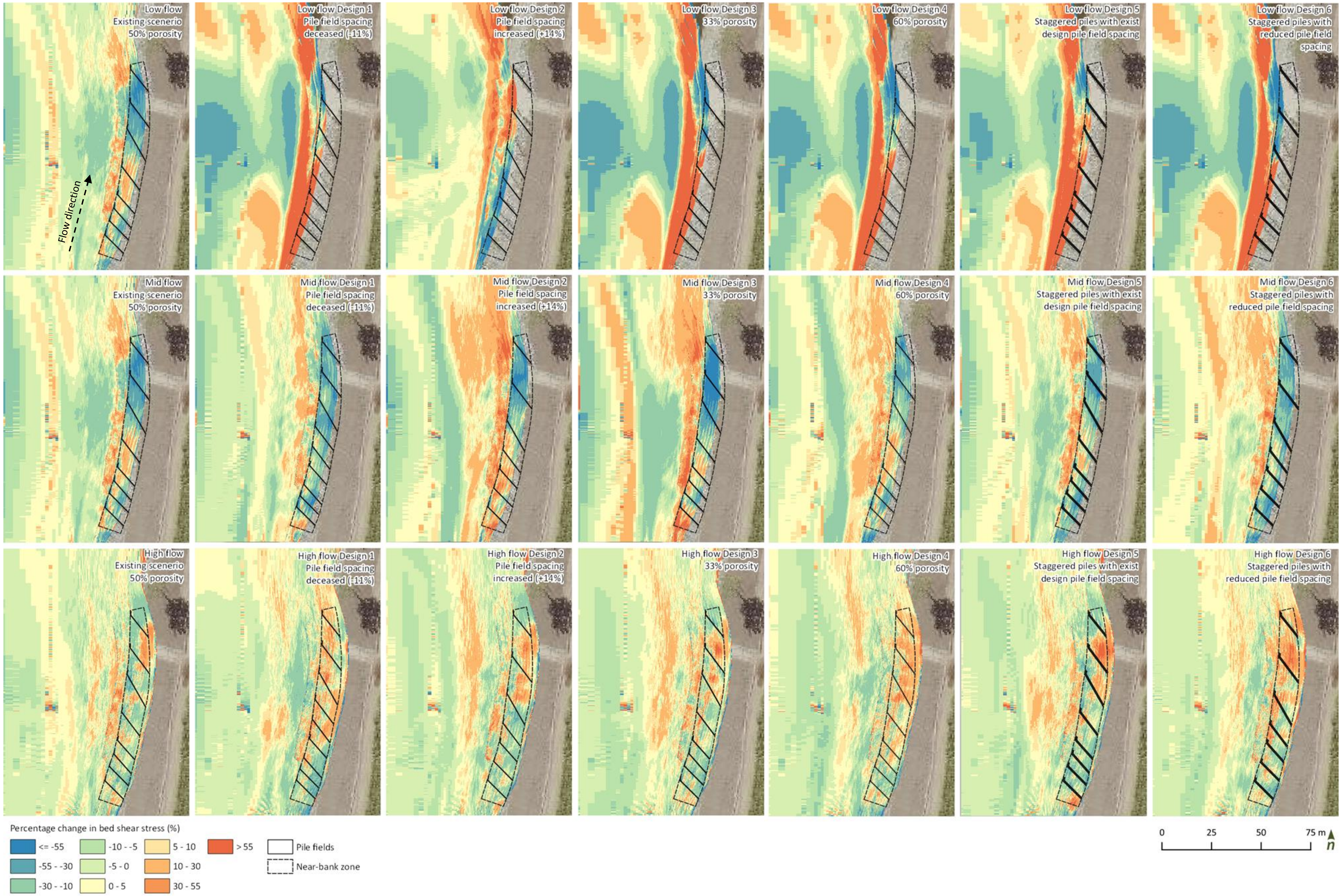

Figure B7. *AEM3D O'Connell River Site 2: percentage change in bed shear stress between the various pile field scenarios and no pile field scenario for the low flow (top row), mid flow (middle row), and high flow (bottom row). As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase. Note: Increase in near-bank shear stress in low flows likely due to water level being close to where there is a drop in the bathymetry which also coincides with the gradient in the dx of the model grid.*

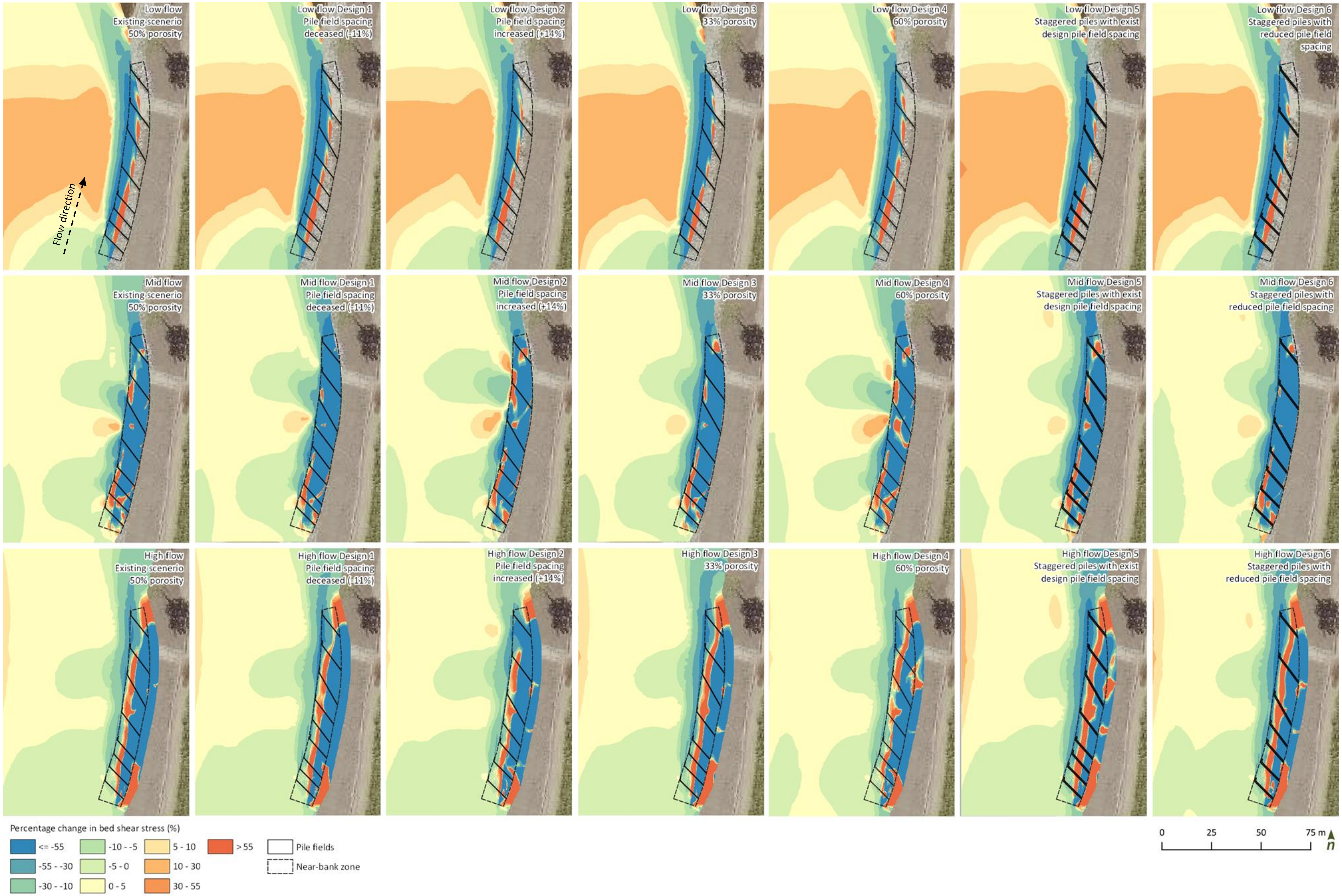

Figure B8. *HECRAS 2D O'Connell River Site 2: percentage change in bed shear stress between the various pile field scenarios and no pile field scenario for the low flow (top row), mid flow (middle row), and high flow (bottom row). As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase.*

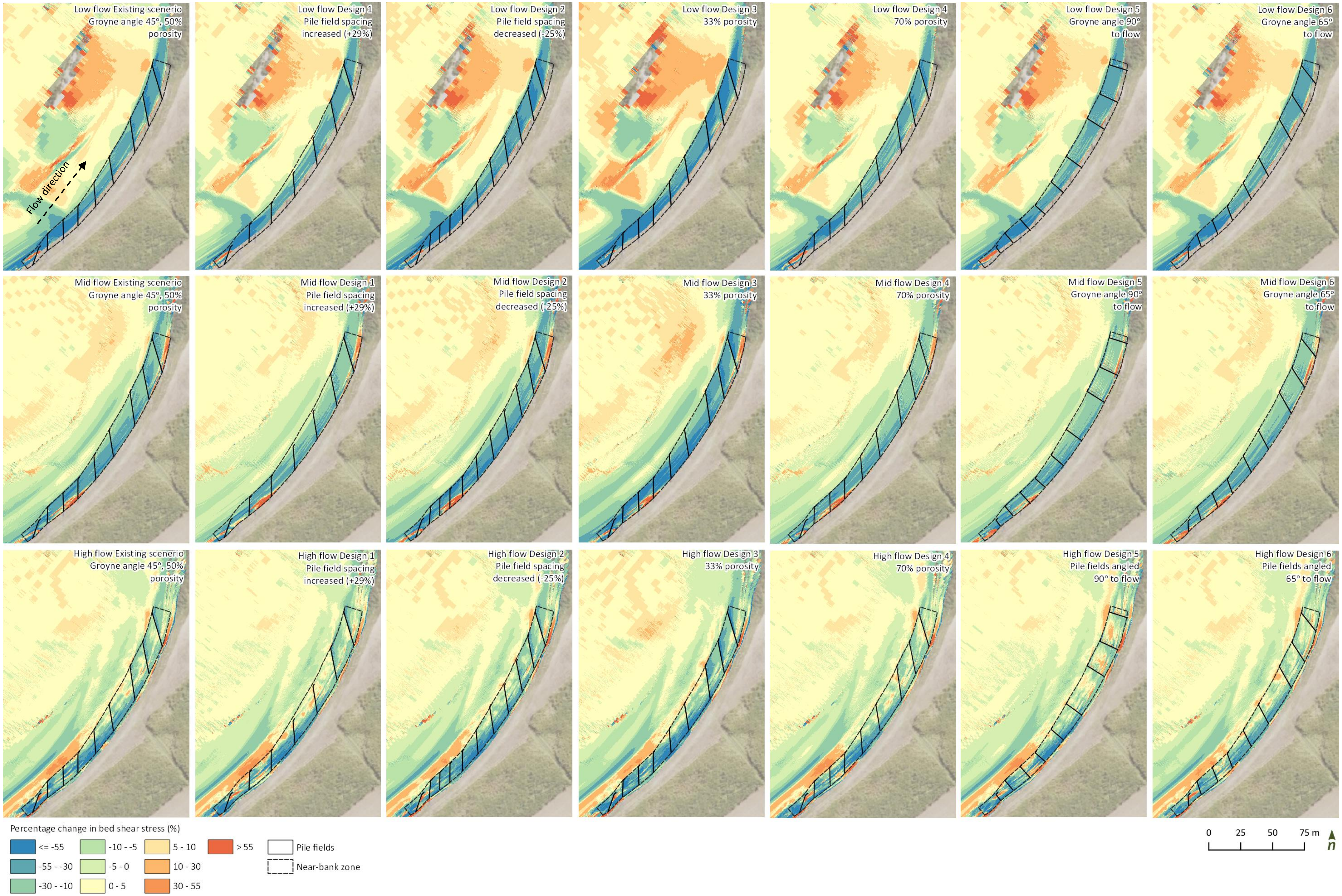

Figure B9. *AEM3D St Helens Creek Site: percentage change in bed shear stress between the various pile field scenarios and no pile field scenario for the low flow (top row), mid flow (middle row), and high flow (bottom row). As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase.*

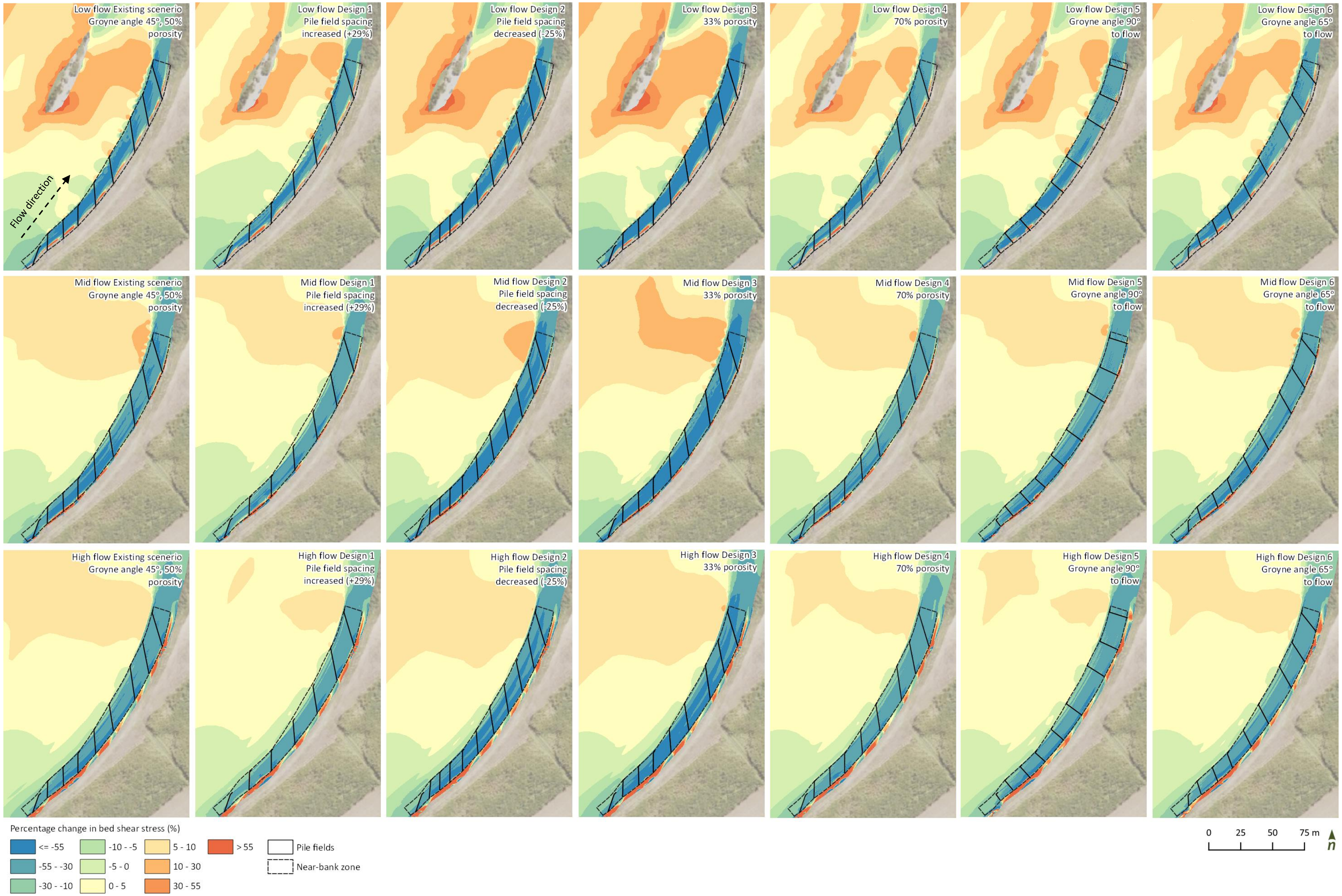

Figure B10. *HECRAS 2D St Helens Creek Site: percentage change in bed shear stress between the various pile field scenarios and no pile field scenario for the low flow (top row), mid flow (middle row), and high flow (bottom row). As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase.*

Figure B11. *AEM3D Molongle Creek: percentage change in bed shear stress between the various pile field scenarios and no pile field scenario for the low flow (top row), mid flow (middle row), and high flow (bottom row). As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase.*

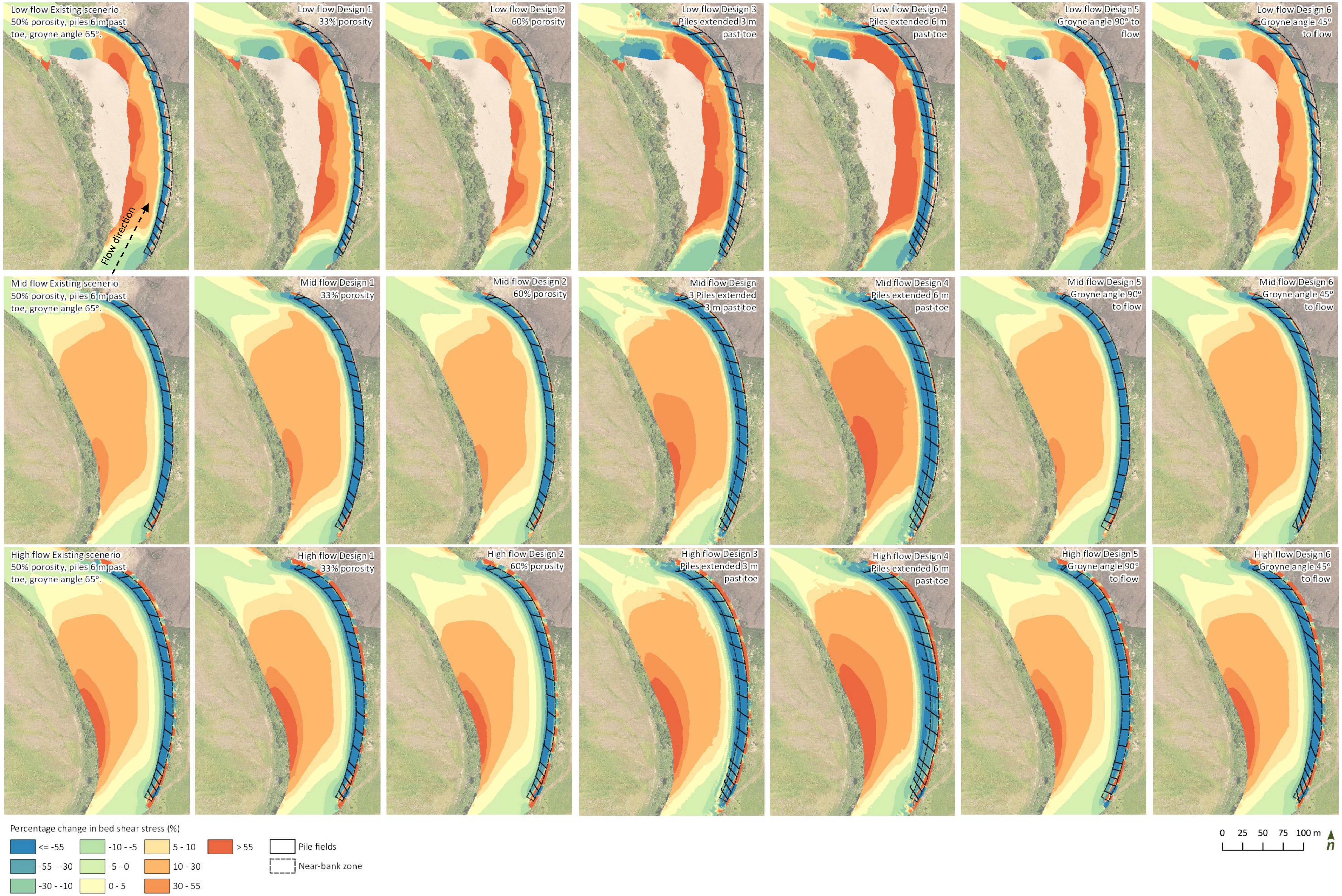

Figure B12. *HECRAS 2D Molongle Creek: percentage change in bed shear stress between the various pile field scenarios and no pile field scenario for the low flow (top row), mid flow (middle row), and high flow (bottom row). As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase.*

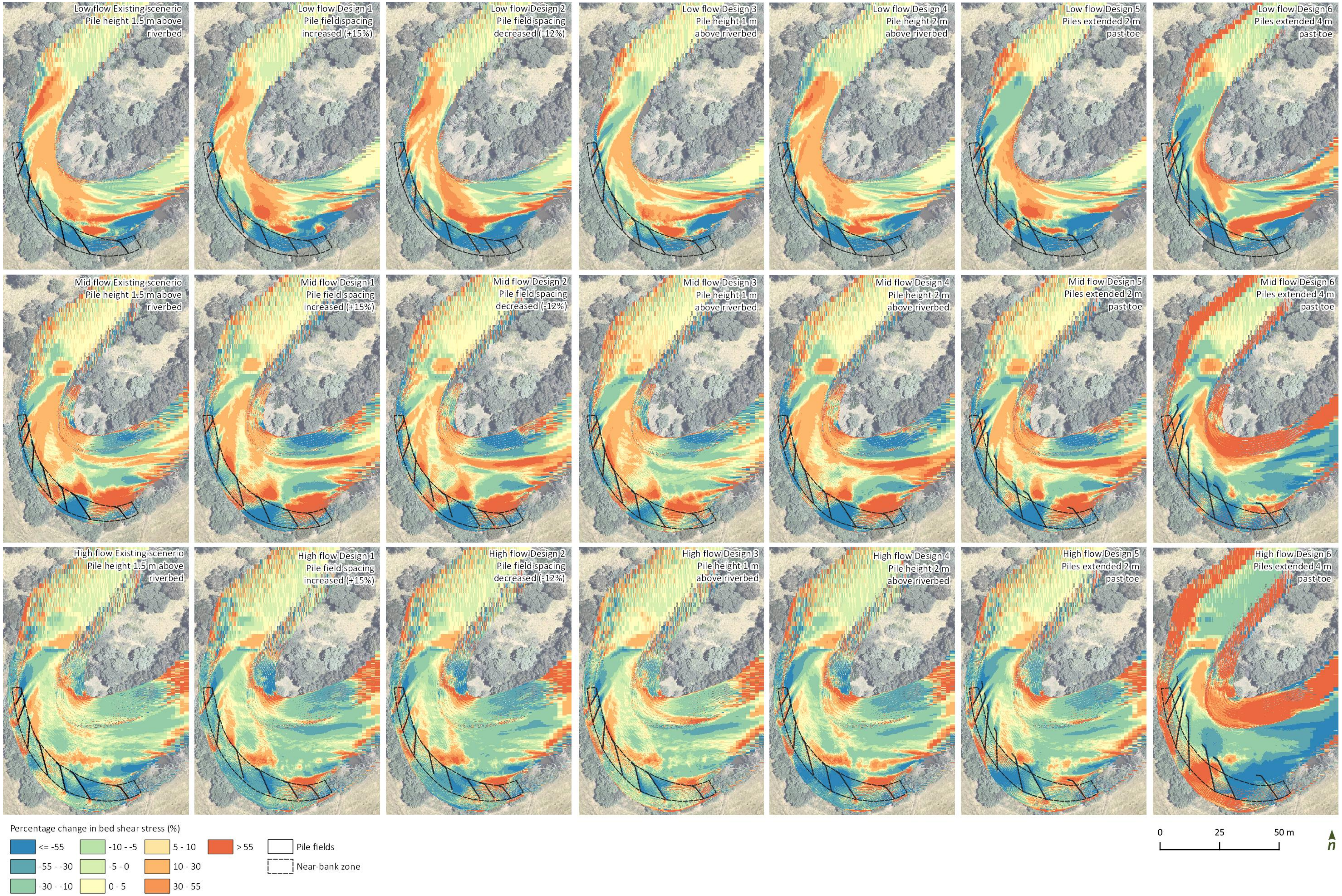

Figure B13. *AEM3D Yarra River site: percentage change in bed shear stress between the various pile field scenarios and no pile field scenario for the low flow (top row), mid flow (middle row), and high flow (bottom row). As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase.*

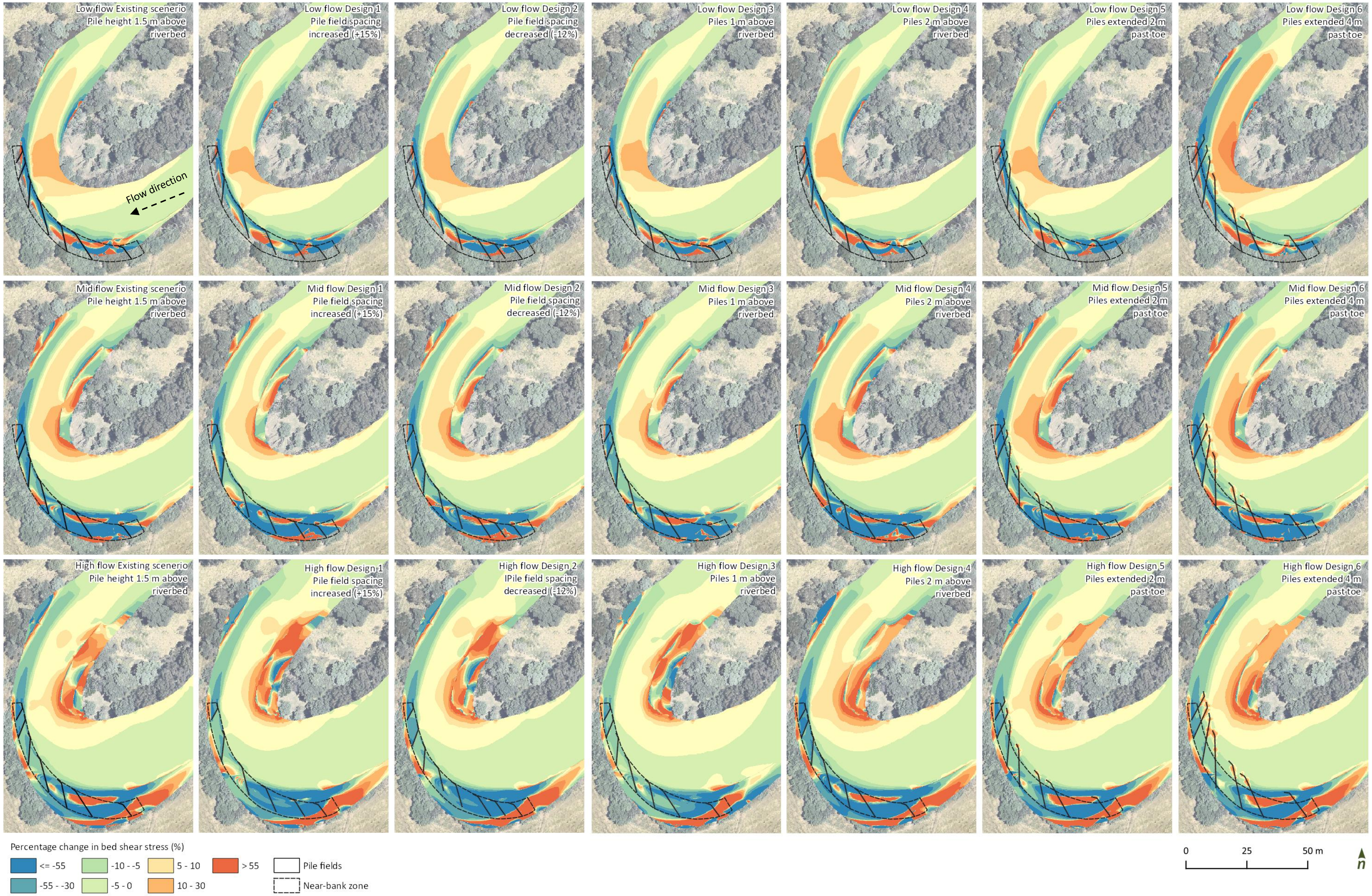

Figure B14. *HECRAS 2D Yarra River site: percentage change in bed shear stress between the various pile field scenarios and no pile field scenario for the low flow (top row), mid flow (middle row), and high flow (bottom row). As indicated, blue/green shades represent a reduction in bed shear stress due to pile fields, whereas yellow/orange shades indicate an increase.*