Auckland Transport

Transport Design Manual

Pavements & Surfacing

(FINAL DRAFT)

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ABBREVIATIONS USED

AADT	Average Annual Daily Traffic
AC	Asphalt Concrete (AC10, AC14, AC20 etc.)
AUSTPADS	Austroads Pavement Analysis Design Software
AGPT-2	Guide to Pavement Technology – Part 2: Pavement Structural Design, Austroads (2017)
BB	Benkelman Beam
CBR	California Bearing Ratio
CDF	Cumulative Damage Factor
CGF	Cumulative Growth Factor
CIRCLY	Mechanistic Pavement Modelling software
CV	Coefficient of Variation
DCP	Dynamic Cone Penetrometer
DESA	Design Equivalent Standard Axles
DfS	Departure from the Standards
DF	Direction factor
DG	Dense Grade Asphalt Concrete
DSL	Design Subgrade level
EA	Engineering Approval Process by the Council for design approval
ESA	Equivalent Standard Axles
FBS	Foamed Bitumen Stabilisation
FWD	Falling Weight Deflectometer
GPR	Ground Penetration Radar
HV	Heavy Vehicles
HCV	Heavy Commercial Vehicles
HVAG	Heavy Vehicle Axle Groups
IANZ	International Accreditation New Zealand
ITS	Indirect Tensile Strength
kph	Kilometer per hour
m	metre
MESA	Millions of Equivalent Standard Axles
MfE	Ministry for the Environment
MoT	Ministry of Transport
mm	Millimetre
MPa	Mega Pascals
MPD	Mean Profile Depth
MSG	Maximum Specific Gravity
NDT	Cumulative heavy vehicle axle groups in the design lane - expressed in HVAG
NHV	Cumulative number of heavy vehicles
NHVAG	Average number of axle groups per heavy vehicle
NZTA	New Zealand Transport Agency (Waka Kotahi)
OWC	Optimum Water Content
PI	Plasticity Index
PIA	Pavement Impact Assessment

RSP	Raised Safety Platform (Raised Speed Table)
PSV	Polished Stone Value
SAC	Structural Asphalt Concrete
SMA	Sone Mastic Asphalt
TAS	Thin Asphaltic Surfacing
TLD	Traffic Load Distribution
TSD	Traffic Speed Deflectometer
UCS	Unconfined Compressive Strength
VDMA	Vehicle Dimensions and Mass
WiM	Weigh-in-motion
WMAPT	Weighted Mean Annual Pavement Temperature

1.0 INTRODUCTION

1.1 SCOPE

The Transport Design Manual for Pavements (TDM as interpreted for this document) is intended to inform the pavement design approach used on the Auckland Transport Road Network and replaces the previous ATCOP version.

The purpose of the TDM is to provide guidance for those responsible for designing and constructing pavements in the region that Auckland Council has jurisdiction over, to allow them to design and construct new pavements that are fit for purpose and meet the Auckland Transport requirements for a low maintenance and safe pavement over the design life. The principles detailed in the TDM can also be applied to the rehabilitation of pavements. The pavement designs should also consider the impacts on stakeholders. This TDM replaces the pavement component of the Auckland Transport Code of Practice published in 2013.

Two pavement design approaches are allowed for in the TDM-Pavement:

1) **Catalogue designs** for which traffic volumes and ground conditions need to be determined, and then designs are selected from a pre-determined range of pavement types matching bands of traffic and subgrade strengths, and
2) **Bespoke designs** for which the traffic volumes and ground conditions need to be determined and then designs are modelled by pavement designers to meet AT design and performance requirements.

Both design approaches are site specific. The **Catalogue designs** can be used without any further detailed pavement modelling, provided evidence is supplied to verify the design subgrade and the design traffic loading. Whereas the **Bespoke designs** require evidence to be supplied to verify the design subgrade and the design traffic, however, the pavement design can be optimised through pavement modelling and material characterisation.

This TDM provides catalogue pavement designs that are suitable for lower traffic volumes up to 5-million ESA. Bespoke pavement designs are required for higher traffic volumes (movements) and the higher place significant roads, as defined by the Roads and Streets Framework which is discussed in Section 1.7 under Project Reliability.

Both the catalogue and the bespoke pavement designs require evidence for the parameters used in the designs.

1.2 PAVEMENT TYPES

Following pavement types are covered in this TDM:

1) Unbound granular pavements, with sprayed chip seal or thin asphalt concrete surfacing.
2) Cement granular basecourse less than 2% of cement, with sprayed chip seal or thin asphalt concrete surfacing.
3) Cement stabilised granular subbase course pavements with unbound or modified granular basecourse, with sprayed chip seal or thin asphalt concrete surfacing.
4) Foamed bitumen stabilised basecourse, with sprayed chip seal or thin asphalt concrete surfacing.
5) Structural asphalt concrete over unbound granular subbase course.
6) Structural asphalt concrete over lean mix concrete or cement bound subbase.
7) Concrete pavement over subbase course (unbound granular, cement stabilised, or lean mix concrete).
8) Block paving use precast concrete
9) Basalt or granite pavers on concrete base

1.3 DESIGN REQUIREMENTS

A pavement design report should be developed by suitably qualified and experienced engineers with a thorough understanding of the Austroads Guides to Pavement Technology series and the associated New Zealand Guides. Of relevance is the latest NZTA Waka Kotahi design standards and guidelines that supplement Austroads. Advice in these documents and the TDM is not to be taken out of context, in addition, it is expected that the latest knowledge, research and updated design documents, will be incorporated into the pavement design process.

For site specific pavement designs (bespoke designs) mechanistic empirical pavement design calculations should be performed on CIRCLY or AUSTPADS using the Drury (decreasing direction) Weigh In Motion (WIM) data, which is available via the following Austroads website link - https://austroads.com.au/publications/pavement/agpt02 and annual weigh-in-motion (WiM) report from https://www.nzta.govt.nz/resources/weigh-in-motion.

If the project is likely to have a truck traffic configuration that differs from the Drury motorway site, then site specific loadings need to be specifically incorporated into the design traffic values. An example of this would be roads in commercial or industrial zones where the predominant heavy vehicles are multi-axle trucks with equivalent standard axles per heavy vehicle higher than the Drury WIM site. Similarly, bus stations or bus lanes, and routes with a large proportion of High Productivity Motor Vehicles (HPMV) may also mean that the Drury WIM data is not appropriate for a specific site. If the site has electric buses using it the pavement needs to be specifically designed to cater for these higher than standard axle weight loadings.

A pavement design report is required to document all the investigations and design assumptions as well as provide details on the pavement modelling (in the case of bespoke pavement design). For bespoke pavement designs, the design report shall be written by a suitably qualified pavement engineer and peer reviewed by a Chartered Professional Engineer (CPEng) whose practice area includes “Pavement Engineering”. This peer review can either be internal or external review.

Should the engineer not be CPEng, a minimum of ten years of pavement design and construction specialisation is required, and they should seek approval from Auckland Council to act as a Reviewer. Approval of an Engineer is at the discretion of Auckland Transport. If the CPEng accreditation is replaced with a new system, then the pavement engineer must meet the requirements of the replacement system.

The pavement design report shall be submitted to Auckland Council for their review and approval under the Engineering Plan Approval (EPA) process. This is discussed in more detail in Section 2.

Drawings shall be provided with sufficient information for construction to achieve the requirements of the pavement design. Minimum requirements are discussed in Section 2.

1.4 DESIGN AND CONSTRUCTION RISKS

Pavement design shall consider the safety in design requirements as detailed in the Health and Safety at Work Act (2015). Auckland Transport may require the designer to demonstrate the safety in design principles used within the design package.

Health and Safety in Design is a process that integrates hazard identification and risk assessment methods during the design process. In pavement design this would include the construction phase, the operation and maintenance phases and ultimately pavement renewal. Treatments with higher initial costs may have significant impacts on long term operational and maintenance costs. These treatments, where required, should be considered in the design phase. This will allow consideration to be given on how to eliminate, substitute, isolate or minimise the risks of death, injury, and ill health to those who will construct, operate, use, maintain, and renew the pavement asset. It will also allow whole of life costs to be better allocated and the best whole of life cost effective treatments identified.

The design process should also consider the pavement interaction with underground services and particularly the impact of stabilising, compaction of lower pavement layers, and dig-outs repairs. The potential impact of these activities should be considered, and the risks managed to preserve the underground services.

Constructability should also be considered during the design process, particularly in rehabilitation treatments where trafficking of the pavement during construction may be required. Consideration should also be given to the traffic delays required by the various construction options; for example, structural asphalt may have higher construction costs but significantly reduce construction duration.

Road maintenance in design considerations shall be included in the assessment of design option attributes. Designs that will require less maintenance intervention in future should be given higher scores in option comparisons.

1.5 AUCKLAND TRANSPORT CLIMATE CHANGE POLICY

1.5.1 Application of Key Policy Area

A major consideration in pavement design is to consider Auckland Transport's climate change policy as detailed in Climate Change Adaptation Policy 2022. Auckland Transport (AT) recognises that changing climatic conditions will significantly impact on the transport system of Auckland including AT controlled operations, services, and assets. Pavement management, new build, rehabilitation and maintenance provide opportunities to put the policy into practice. However, environmental considerations (whole of life!) to be determined in pavement design. Developers will need clarity and explicit guidance on how this should be assessed, because whole of life considerations may demand a higher capital cost pavement with lower future maintenance costs.

The AT approach to climate change is applied in accordance with the following principles:

Manage the impacts of climate change.

Reduce the vulnerability of assets exposed to climate change and prioritise the risk management of assets so that transport services can continue if disruption occurs.

Our response is equitable

Climate adaptation considers economic, social, and environmental issues equitably, and balances the needs of present and future generations.

Prioritise activities

Understand where infrastructure assets and their services are exposed and vulnerable to climate impacts and consider the capacities and opportunities of AT, suppliers, and the community to act. Maximise the renewal programmes to improve adaptive capacity.

Minimise environmental and biodiversity impacts

AT adopts a precautionary approach towards proposed activities with effects on the environment and indigenous biodiversity that are uncertain, unknown, or little understood but potentially significant.

Our response is integrated

Our response to the changing climate is a dynamic and iterative process that is an integral part of all AT's organisational activities, and is part of the organisation's purpose, governance, leadership and commitment, strategy, objectives, and operations.

Our decision making is robust

Consider long-term climate impacts when we design and invest in infrastructure, so the right infrastructure is in the right places. Manage risk by making decisions despite uncertainty, using the right tools, guidance, and methodologies to manage climate risks. Allow for uncertainty when planning for future risk.

The application to pavement design and pavement management will include (but is not limited to):

- Environmental considerations (whole of life!)
- Choice of pavement
- Reuse of materials

- Construction methods
- Quality assurance
- Energy saving
- Weighted Mean Annual Pavement Temperature (WMAPT): AT has considered the effects from Climate Change for the temperature adjustment factor. Due to potential increasing temperatures, and higher temperatures associated with increased high-density housing in the urban environment, the Weighted Mean Annual Pavement Temperature (WMAPT) or operating temperature to be used for design is **25°C** for both pavement rehabilitation and maintenance and new pavements. Further details are provided in Section 6.3.
- Durability
- Sustainability more on this in the next section and in Section 9.2.

1.5.2 Climate Change Adaptation for Pavements

NZ has experienced extreme weather events over the last few years that have confirmed that Climate Change can, and is, producing a wide range of impacts on infrastructure. Transportation networks have been impacted by extreme weather events. Much has been written about climate change and its impact on transportation systems, and literature is now emerging on how climate change specifically affects pavement systems, and what adaptation strategies might be pursued. However, the current practice for design and construction of pavements is largely limited to general observations and is lacking specific adaptation strategies. This section provides an overview of Climate Change elements and specific impacts on pavement, with applicable and practical pavement adaptation strategies that can be implemented now and in the future.

This applies to flexible and rigid (concrete) pavement, including the surface wearing course, underlying pavement layers and any subgrade treatment. This section does not address climate change adaptation issues that are beyond the scope of pavement systems, such as identification of vulnerable routes, of both road and bridges, due to storm surges or sea level rise, and protection of pavement systems against extreme weather events as follows:

Temperature Impacts

- General increase in temperature. Most models project some increase in average air temperature, with some predicting as much as 2.2 °C.
- Higher extreme temperatures. As average temperatures increase, an increase in the frequency and duration of extreme temperatures is also projected. Consequently, weighted annual mean pavement temperature (WAMPT) is also expected to increase.

Precipitation Impacts

- Increases in average annual precipitation.
- Wetter winters and drier summers.
- Increased intensity of rainfall, resulting in higher runoff and flooding of low-lying areas.
- Storms and cyclones are becoming more powerful.

Sea Level Impacts

- Sea levels have been rising since the early 1990s and will continue to do so. Projections on the amount of sea level rise are for up to 2m by 2100.
- Sea level is also affected by wind strength, wind direction, water currents and air pressure variance.

Mitigation measures for road networks are required to be implemented to protect transport system reliability.

1.5.3 Impacts on Transport Systems

Many government agencies, local authorities and private organizations are planning for Climate Change. This planning considers the impacts of Climate Change, the necessary adaptations to minimise or mitigate the impacts. An example is adapting to a lower embodied carbon in development of transport infrastructure.

Auckland Council has developed and adopted a Climate Change Policy to address the impacts and is beginning to explore how to adopt current practices to mitigate the impacts. Climate change impacts are expected to increase costs for the users of, and the construction, maintenance and operation of, the transportation system. The following climate change impacts are expected to affect transport infrastructure systems:

- Extreme weather events, with increased levels of precipitation, and cyclonic conditions, are affecting the reliability and capacity of the transportation system.
- Sea level rise with storm surges continue to increase the risk of major coastal impacts on transportation infrastructure, including both temporary and permanent flooding of roads, ports and harbours, railway lines, tunnels, and bridges.
- Higher temperatures lead to softening and rutting of asphalt, chip loss and/or bleeding of bitumen in chipseal surfacing.

Climate Change impacts on transport infrastructure are increasing. Work on specific adaptation strategies to address these impacts is currently general in nature.
Tables 1 provides Potential Impacts of Climate Change and Table 2 includes potential adaptations to mitigate identified impacts.

Table 1: Potential Impacts of Climate Change

Climatic Event	Impacts of Current Climate Variability	Examples of Potential Vulnerabilities
Increasing temperatures and periods of extreme heat.	Causes road surface shape loss, and cracking. Increased thermal movement in concrete structures. Causes rail deformation and potential derailment. Changes in design procedures to accommodate increased heat. Potential delays in construction of transportation infrastructure.	The possibility of increased future need for road maintenance and road closures for heat- related problems may impact road and rail system reliability. Busway / Rapid Transit Routes may use more energy to maintain passenger services at a comfortable level. Rail lines may require installation of more heat-resistant tracks or face periodic cancellation of services due to derailment risk.
Flooding events, and increased intensity of rain event / storms	Can lead to: ▪ submerged roads ▪ flooded underpasses ▪ scour of roads and bridges ▪ landslides ▪ overloading of drainage systems ▪ compromised structural integrity of roads, bridges, and tunnels ▪ adverse impacts on pavement layers ▪ a need for larger waterways ▪ road and rail closures ▪ increased maintenance costs ▪ Increased operational costs.	Low lying rail and road network areas. Emergency response vehicles are unable to attend incidents in a timely manner. Transport corridor customer and stakeholder delays and costs. Low public perception of Transport Asset Owner ability to manage extreme events. Insufficient available maintenance crews and equipment to respond in advance of extreme events to maximise capacity of stormwater system inlets and pipes before events occur.

1.5.4 Pavement Adaptation Strategies

The development of pavement distress will increase over time due to the impacts of Climate Change. The adaptation methods are within the range of current material properties. However, because new and rehabilitated pavement designs are for a 25-year Design Period or longer, the key adaptation issues will pertain, not on how to deal with potential impacts, but rather on when to modify current design and maintenance practices.

The focus of these efforts is to integrate Climate Change into current pavement design methodologies and predict pavement performance based on future Climate Change scenarios are provided in Table 2.

Recommended immediate changes in design and construction practice, with minimum requirements under this TDM are provided in Table 3.

Table 2: Potential Impact and Adaptation for Pavement Design

Climate Change Impact	Adaptation Strategies

Higher / Extreme Temperatures	Flexible Pavement:
	▪ More rut resistant asphalt surfacing by adopting:
	  o Stiffer bitumen binder grades provided asphalt fatigue performance is not compromised
	  o Increasing the use of polymers in asphalt binders
	  o Increasing use of stone mastic asphalt, or other high rut resistant mixes
	▪ Increasing use of stiffer pavement layer designs, to reduce rutting risk
	▪ Change to less heat absorbent surfacing, a move away from black asphalt to lighter coloured surfaces that reflect heat back, e.g. concrete, particularly in cities where heat gain is becoming a problem, requiring planting trees in the road corridor
	▪ Increased use of binders that age more slowly, or more use of asphalt pavement preservation techniques such as emulsion rejuvenation sprays to address binder aging

	Rigid Pavement: ▪ Greater consideration of concrete coefficient of thermal expansion and curling ▪ Incorporation of design elements to reduce damage from thermal effects including closer joint spacing and/or thicker slabs
Extreme Rainfall Events, Sea Level Rise, Coastal Inundation	▪ Increased need for surface friction meaning potentially more focus on surface texture and maintaining adequate skid resistance ▪ Design to consider an increase in minimum cross slopes to facilitate flow of water from surface ▪ Design to increase resistance to rutting due to saturated subgrades ▪ Design to mitigate the risk of a reduction in the structural capacity of unbound aggregate bases and subbases due to saturation becoming more common ▪ Design to consider use of porous surface mixtures ▪ Increased need for surface drainage to prevent flooding with more catchpits, larger stormwater network pipes, and larger culvert capacity ▪ Design may require increased stormwater detention areas to mitigate increases in runoff peaks ▪ Increased need for adequacy of design, installation and maintenance of sub-surface drainage and stormwater system ▪ Where feasible raising pavement sections ▪ Design roads to be relocatable in the event of inundation by selecting materials that can be economically reclaimed and reused ▪ Design and use of improved visibility pavement delineation marking ▪ Investigation to determine embankment stability in high levels of precipitation and design of improvements to mitigate risk of embankment

Table 3: Proposed Target for Pavement Materials and Design

Item	Description	Minimum Target	Clarification
1	Using rapid breaking Emulsion for membrane sealing and resurfacing of road instead of cut-back bitumen	50% of all surfacing and gradually increase to 100% within 5 years	All membrane sealing is currently emulsion, which can be sprayed at low temperature.
2	Use of Reclaimed Asphalt Pavement (RAP) blending with natural aggregates and virgin binder	15% RAP in all mixes.	AT has approved asphalt concrete mixes with up to 30% RAP using both extracted binder and aggregates
3	Increase weighted annual mean pavement temperature (WAMPT) for adjusting the resilient modulus (determined from the MATTA test) to be used in the pavement design and modelling.	WAMPT 25 degree Celsius for both new pavement and renewal / rehabilitation and resurfacing for existing roads	Austroads AGPT02:24 has adopted WMAPT of 24 degrees Celsius in 2004 for Auckland. NZTA M10-2020 proposed WAMPT 23deg Celsius. Increasing WAMPT is pragmatic for new and existing roads.
4	Use of Eco/ low carbon concrete for concrete used in road pavements, shared use paths, footpaths, cycleways, vehicle crossings, kerbs, nibs and channels.	25% less embodied carbon concrete for non-structural pavement.	A new Practice Note is being published in TDM on low carbon concrete – design, testing and construction.
5	Use of recycled crushed concrete and crushed glass aggregates for pavement construction	Suggest having at least 25% on all construction, however both reliability and quality of supply would most likely not be an issue.	AT 800 Series Aggregate Specification allows recycled crushed concrete and recycled crushed glass aggregates
6	Adopting foamed bitumen modified or cement modified (less than 2% cement), or cement bound (3% or more cement) for pavement rehabilitation and strengthening, instead of full depth reconstruction with imported pavement materials.	Currently no target. Bitumen and cement bound basecourse requires a change in design philosophy and crack mitigation techniques.	Retention of existing pavement materials for pavement strengthening reduces the use of less imported pavement materials.
7	Use of geosynthetics (geogrid / geotextile and geocell) in road pavements to reduce the overall pavement layer depth.	Currently no target. This can also be used for undercutting poor subgrade conditions to facilitate constructability.	Use of geosynthetics with proper design may reduce pavement depth and thus reduce embodied carbon.
8	Unsealed roads are vulnerable to get washed out on heavy rainfall / storm event	No unsealed roads are built, and existing unsealed roads must be upgraded with sealed surfacing in the network.	Seal extension to provide to all rural roads to reduce maintenance cost of renewal with basecourse aggregates.

1.5.5 Pavement Design Approach and Flexibility

1.5.5.1 Asphalt Concrete Pavement

It is essential that current design practices have a shift in focus, from current pavement temperatures to those expected in 15 to 20 years.

Designs consisting of thick asphalt layers (e.g. long-life perpetual pavements) composed of multiple lifts with varying levels of stiffness can provide greater pavement durability. The asphalt pavement layers are largely moisture insensitive, and the surfacing (which is most dramatically affected by temperature) levels of service can be maintained through resurfacing when required. Thus, over time the surfacing design can adapt to Climate Change impacts and be readily modified without affecting the underlying structure.

Increasing temperatures can lead to surfacing more susceptible to damage, a shorter pavement surfacing life, and additional pavement life-cycle costs. To mitigate these Climate Change issues, such as rutting or flushing due to increased temperature, higher Performance Grade binders can be adopted for asphalt layers. Design can adopt an increase in weighted mean annual pavement temperature for deriving fatigue parameters.

A strategic change to more rut resistant asphalt, such as stone matrix asphalt (SMA) and asphalt mixes using EVA type (plastomeric) polymer-modified binders for increased rut resistance. Decreasing asphalt binder temperature susceptibility through increased polymer binder use will enhance mixture stability under higher temperatures.

The use of warm mixed asphalt (WMA) saves energy in asphalt production and therefore lowers embodied carbon.

Table 4 provides approaches to improving sustainability for asphalt pavements while reducing greenhouse gas (GHG) emissions during production and construction and reducing embodied carbon.

Table 4: Approaches to Improving Pavement Sustainability for Asphalt Surfacing and Pavement

Asphalt Materials Objective	Sustainability Improving Approach
Reduce Virgin Bitumen Binder Content in Asphalt	Use RAP as partial replacement for asphalt binder if the same or better performance can be realised
	Use recycled rubber as a polymer in asphalt binder
	Use bio-binders such as lignin in place of bitumen
	Use sulphur-modified asphalt.
Reduce Virgin Aggregate Content in Asphalt	Use greater quantities of RAP as an aggregate source if the same or better performance can be realised. However, for thin asphalt surfacing on unbound granular or modified granular basecourse, check RAP addition does not create a surfacing too stiff that will crack due to fatigue earlier than a lower modulus asphalt.
Reduce Energy and Emissions to Produce Asphalt	Employ new, more efficient plant designs to reduce energy consumption, and embodied carbon.
	Use warm mix asphalt.
Extend Life of Asphalt	Design for perpetual pavements where appropriate
	Use WMA to reduce mixing temperatures.
	Improved mixture designs with performance testing such as beam fatigue testing and flexural modulus testing. Some cost for new equipment will be required, training, payback from longer lives.
	Use polymers and other additives.
	Use epoxy modified binder to improve surfacing life where micro-texture skid resistance requirements can be maintained.
Reduce Materials Transportation	Use more locally available materials.
Reduce the Thickness of Asphalt	Use of high modulus asphalt concrete (EME2) more often for thick asphalt below surfacing where construction / production is viable
Extend Lives of Seal Coats	Use polymer or epoxy binders.

Asphalt Materials Objective	Sustainability Improving Approach
Reduce Need for Virgin Materials and Transportation	Use in-place recycling (full depth reclamation, partial depth recycling) – foamed bitumen treatment or cement stabilisation to modify pavement under an asphalt surfacing. May have high construction variability.

1.5.5.2 Foamed Bitumen Modified or Cement Bound Stabilised Pavement

Conventional unbound granular subbase and base materials are known to be susceptible to changes in moisture condition, becoming weaker and less stiff as their moisture content approaches saturation. Foamed bitumen and cement bound stabilised subbase and basecourse materials are more resistant to loading when the pavement is saturated and can be used in areas where periodic inundation is expected to increase.

1.5.5.3 Concrete Pavement

Concrete pavements can offer robust long-lasting designs. Incorporating low shrinkage concrete, moisture insensitive supporting materials, and good load transfer at any joints will provide a long life or perpetual pavement.

If skid resistance is required, concrete pavements can either be designed with an additional thickness that will accommodate multiple future diamond grinding, or can be covered with a thin surfacing, to provide the necessary functional level of service requirements such as micro-texture, macrotexture, spray mitigation and noise mitigation.

Modern concrete technology such as steel fibre reinforced concrete, and roller compacted concrete can allow jointless pavements. Roller compacted concrete can significantly decrease the embodied carbon in concrete pavements due to the relatively low cement contents and not requiring any reinforcing steel or dowelled joints.

Concrete has a major advantage in urban environments in that a lighter colour than asphalt can reflect heat rather than absorb it. With increasing temperatures, the use of concrete is a mitigation strategy for the risk of over-heating of city areas.

It is noted significant technical and performance advantages to cast-in situ concrete having a lower water/cement ratio resulting a higher strength/cement content ratio and low shrinkage not requiring reinforcing steel or fibres including reduced curing times; higher durability reaching concrete strength more than 40MPa crushing strength, lower embedded carbon compared to conventional concrete, early traffic access to pavement, and faster with construction.

1.5.6 Changes in Construction Activity Periods

Most transportation organisations place weather limitations on pavement construction. Usually, these limits involve:

a) Minimum surface temperatures for asphalt paving,
b) Protection from extreme temperatures and dry and windy conditions during stabilisation of subgrade and modifying aggregates, concrete placement and curing, and
c) Prevention of paving during heavy rainfall.

Consideration should be made to modifying specifications for paving works due to seasonal weather changes. Climate Change impacts such as a general increase in temperature, higher extreme temperatures, changes in annual precipitation, and increased precipitation intensity may impact when pavement works can be undertaken, potentially extending the construction season in some cases or decreasing allowable days for construction in other cases. Traditional cold and wet weather months have been used as a basis for chip sealing seasons starting in summer and finishing in autumn. Climate Change is likely to result in a change to the permissible windows for chip sealing, with warmer winters and wetter summers.

1.5.7 Pavement Resilience in Extreme Weather Events

Given the recent frequency of extreme weather events, an area of increasing concern is the ability of transport infrastructure to withstand such events in a serviceable condition.

Pavements submerged due to storm events tend to lose strength. Moisture susceptible materials within and underlying the pavement structure quickly come to equilibrium saturated conditions during inundation.

Recommended adaptation strategies include the use of moisture insensitive materials such as cement bound or foam bitumen stabilised pavement materials to replace unbound granular materials in the pavement structure. This minimizes the reduction in structural capacity that occurs when pavements are inundated.

1.6 SUSTAINABILITY

Auckland Transport has a duty to promote and encourage sustainable construction in line with the NZ Government's and Auckland Council goal of reaching a net zero carbon output by the year 2050. Sustainable considerations involve understanding the cradle to grave development of all pavement and drainage materials, the initial construction as well as future maintenance and renewal needs of each road asset, and which pavement materials fit within a circular economy. In addition to Section 1.5 Pavement Adaption Strategies due to Climate Change, the pavement design process should align with the following sustainability outcomes:

- Target zero waste through waste minimisation and/or recycling/re-use;
- Energy savings in construction;
- Reduction in contribution to greenhouse gases production;
- Contribution to whole of life cost reductions for maintenance of road transport assets;
- Usage of biofuels and other similar fuel derivatives other than fossil fuels; and
- Use of pavement materials that fit within a circular economy (material does not end up as waste).

Given the numerous existing and potential sustainable inputs into road pavements, this might include consideration of some of the following:

- Use of recycled materials in road construction, such as:
  a) Asphalt, in the production of recycled asphalt pavement (RAP) mixes already permitted by existing NZTA specifications.
  b) Aggregates, with the use of recycled crushed concrete for unbound pavement construction and re-processing of waste aggregates and the use of industrial by-products (e.g. slag in asphalt).
  c) The use of existing pavement materials in construction, such as subgrade undercut situations where a variety of suitable materials could be made available, possibly incorporating stabilisation.
  d) Examples: of recycled materials can include glass, sand, millings (environmental concerns may need to be addressed) or other recycled materials may be suitable.
  e) Low carbon concrete GBC cement and slag with EC rating minimum 25% or more.
  f) Mechanical or chemical stabilisation, rather than removal of, existing subgrade material.
- The sourcing of locally available materials to reduce the transport distance for the materials.
- The use of stabilisation / modification of pavement aggregate is promoted within the provisions of NZTA specifications such as NZTA M4 and AT 800 Series Aggregate Specification making locally available aggregates suitable for use as basecourse material.
- The use of slightly lesser grades of aggregate is suitable for use in some applications where the use of premium aggregate is unnecessary. Some provisions have been made in this document to allow this. Refer to At TDM Section 3: Specifications for Transport Infrastructure, Series 800 for Aggregate Supply.

- The stabilisation of existing road materials, including:
    i. In-situ stabilisation, which incorporates the current use of some recycled materials or by-products as additives.
    ii. Designing pavements to allow future in-situ stabilisation.
    iii. Processing and treating of pavement aggregates using pug mills (a machine in which clay or other materials are extruded in a plastic state).

Notwithstanding the above, all pavement materials should meet the AT TDM Section 3: Specifications for Transport Infrastructure, Series 0800 Specification for the Supply of Aggregates (draft), which specifies the minimum aggregate standards for the AT network. If a lesser grade material is proposed by the designer and the material does not meet this Specification, then a Departure from Standards (DfS) shall be applied for to seek possible approval from Auckland Transport and shall be obtained before the material is used on site.

The NZTA *Technical Advice Note #21-07 Moving from Hot Cut-Back to Bitumen Emulsion* should be considered, and bitumen emulsions should be used. Similarly, in-situ recycling of pavement materials using stabilising agents such as cement and or foamed bitumen or other method should be considered as a first option for pavement rehabilitation. The NZTA M4 specification allows for the incorporation of crushed glass or recycled concrete into the basecourse, and these options should be considered for pavement basecourses or subbases. The NZTA M10 specification allows for the incorporation of recycled asphalt into the asphalt and, where possible, recycled asphalt should be considered.

1.7 PROJECT RELIABILITY

The Roads and Streets Framework (Auckland Transport 2020) is used to categorise the expected traffic loadings and therefore inform the treatment options for a particular site. In particular, the movement significance as defined in the Roads and Streets framework will be used to inform the design treatments and acceptable risks for individual roads.

- M3 roads have high strategic importance with higher volume of users and so lower risk pavement designs are appropriate for these roads.
- M2 roads have medium strategic importance so some greater risk than M3 associated with pavement failures is acceptable.
- M1 roads have lower strategic importance and can tolerate treatments that may require more ongoing maintenance.

The place significance, while having a lesser influence on actual pavement design will inform the acceptable risk profile.

- P3 roads attract activity from across the region and even from across New Zealand and international.
- P2 roads attract activity from across a sub-region or neighbouring local board area.
- P1 roads perform predominantly a local function with a small catchment of users.

While much of this TDM Section is applicable to both new designs and pavement rehabilitation designs, specific guidance on rehabilitation designs is provided in Section 6.2.

The pavement design reliability, when designing according to the NZ Guides and Austroads, shall meet the values defined in Table 5 for both flexible and rigid pavement design. The Place Significance, P1, P2 and P3 and the Movement Significance M1, M2, and M3 are defined in the Roads and Streets Framework (RASF) published by Auckland Transport 2020.

Table 5: Design Reliability required for Various Combinations of Movement and Place Significance

P1/M3 - 95% (Primary Collector Roads)	P2/M3 - 95% (Arterial / Collector with Bus routes)	**P3/M3- 95% (Regional / Strategic Routes)**
P1/M2 -90% (Residential Streets)	P2/M2- 95% (Industrial / Commercial Roads)	P3/M2- 95% Secondary / District Collector Roads)
P1/M1 - 90% (Accessway / Local Roads)	P2/M1- 90% (Residential Streets)	P3/M1- 95% (Bus Routes)

Under this requirement, design reliability for both flexible and rigid pavement design based on standard road classification will be as follows:

- Arterial, Principal, Regional and Regional Primary and Secondary Collector roads including bus routes shall be assigned 95% design reliability.
- High productivity mobility routes (HPMV) in AT networks as published by Waka Kotahi should also be considered M3 movement and assigned 95% design reliability.
- All roundabout and intersections to be assigned to 95% reliability.
- Signalised or non-signalised traffic intersection and roundabouts assigned 95% design reliability.
- Local roads with no bus route shall be assigned 90% design reliability.
- All rigid pavement - concrete carriageway, port and bus terminal shall be assigned 95% design reliability.

1.8 CATALOGUE OR BESPOKE DESIGN SELECTION

For the movement classifications P1/M2 (Residential Streets), P1/M1 (Accessway / Local Roads) and P2/M1 (Residential Streets) traffic classification, the design can follow the catalogue design approached detailed in Section 5.0 of this Transport Design Manual. This catalogue design is limited for up to 5 million ESA for traffic loadings as provided in Table 6: Suitable Pavement Designs for Various Traffic Levels.

While Section 5.0 provides CATALOGUE pavement designs that can be adopted, the designer is free to develop a site-specific BESPOKE pavement design as detailed in Section 6.0 provided there is sufficient data to support the design approach.

Bespoke designs are required for P3/M2 (Secondary / District Collector), P2/M2 (Industrial / Commercial Roads) and P3/M3 (Primary Collector) P2/M3 (Arterial / Collector, Bus routes) classification roads.

Supplementary information for catalogue design and bespoke design should be conducted according to NZ Guide to Pavement Structural Design and Pavement Evaluation and Treatment Design and the Austroads Guide to Pavement Technology: Part 2 and Part 5.

1.9 DESIGN APPROVAL

When submitting a pavement design for approval by Auckland Transport the pavement design should demonstrate the design has been completed in accordance with the latest versions of TDM Standards and Guidelines, the latest NZTA Waka Kotahi Design Standards and Guidelines as a supplement to Austroads, including Austroads Guide to Pavement Technology documents and other industry relevant specifications. It has been informed that the current NZTA Pavement Structural Design Guides are changing to a Design Standard in 2026.

The pavement design shall be submitted through the Te Kaunihera o Tāmaki Makaurau/Auckland Council Engineering Plan Approval (EA) which can be found on their website.

2.0 PAVEMENT DESIGN

2.1 GENERAL

Pavement design has been divided into two categories as listed below. For local roads or with traffic loading up to 5 million ESA, designers can choose to adopt a Catalogue Design not requiring mechanistic pavement modelling and other Engineering Approval (EA) compliance checks in the design.

1) **CATALOGUE Design** – A typical pavement design profile provided in this TDM prepared for a range of pavement types and based on a range of subgrade conditions and traffic loading combinations.
2) **BESPOKE Design** – A project site specific custom pavement design including mechanistic pavement modelling for both design traffic loading and compliance with AT TDM deflection and curvature functions.

2.2 PAVEMENT DESIGN REPORT

A pavement design report is required for both catalogue and bespoke designs. The pavement design report should comprise all the information necessary to support the design selected.

Design assumptions will be documented so that Auckland Transport has sufficient information to review the design. In particular, the pavement design report will discuss:

- Determination of the subgrade CBR (Section 3.0)
- Pavement investigation & design approach (Section 3.0)
- Design traffic load calculation (Section 4.0)
- Catalogue Pavement Design (Section 5.0) or
- Bespoke Pavement Design (Section 6.0)
- Material characterisation for pavement modelling (Section 6.3)
- Selecting Surfacing (Section 7.0)
- Construction requirements (Section 8.0)
- Construction quality assurance (Section 9.0)

The following sections detail some additional considerations that should be included in the design report; however, the list is not exhaustive and other design considerations are likely and should be included in the report.

2.3 PAVEMENT DESIGN DRAWINGS

Project Drawings shall include sufficient information to construct the pavement assets in accordance with the design and AT TDM requirements. Minimum details to be included are:

- Plan to identify the different colour-coded areas of pavements and all surfacing types.
- Pavement cross sections showing the pavement layer types, layer depths, and design subgrade CBR strength.
- Notes requiring testing on site to confirm design subgrade strength after cut and fill operations, including methodology for testing and values associated with various subgrade strengths.
- Reference to material specifications, e.g. AT TDM Section 3: Specifications for Transport Infrastructure, Series 0800 – Specification for the Supply of Aggregates, and labelling pavement layers to match.
- Binder requirements for asphalt layers in accordance with NZTA M01-A.

- Skid resistance site category and design traffic (ESA) to enable selection of surfacing sprayed chip seal or asphalt aggregates in accordance with NZTA T/10 aggregate performance method.
- Minimum macro texture requirements for asphalt surfacing at end of defects liability period.
- Design strength of concrete layers where not included in a TDM standard detail.
- Concrete pavement shrinkage control steel, sawn joint spacing and construction details, expansion joint spacing and isolation joint construction details.
- Deflections in millimetre (mm) to be achieved on top of the basecourse layer before thin surfacing is applied.
- Contingency design(s) for weaker subgrade strengths than the design strength to ensure the Contractor has guidance without delay on the Drawings as to changes required in such weaker areas.
- Drawing details and template to be as per Standard AT TDM where appropriate.
- Long section details of transitions between pavement types (new to new or new to existing), to show staggered layer interfaces.
- Plan view line to show location of subsoil pipe drains, flow direction, and invert levels at connection points to stormwater assets. Invert levels can be shown on stormwater long sections if preferred.

2.4 DETERMINATION AND ASSESSMENT OF SUBGRADE STRENGTH

The subgrade strength is to be confirmed prior to submission of the pavement design. The report should document how the design subgrade value or values have been determined. Supporting field investigation and laboratory testing should be provided in the appendices. While material and subgrade strength assumptions may be appropriate for cost estimations and preliminary assessments, field, and laboratory testing at and below the design subgrade level must be undertaken prior to detailed design required for submission of Engineering Approval. Further requirements are detailed in Section 3.0.

2.5 UTILITY PILOT TRENCHING

Utility service as-built information, Before-You-Dig (B4UDig), AT RAMM database and Auckland Council GIS System, and other utility company information sources should be used to identify the likelihood of underground services at the design stage. This especially applies for existing pavement rehabilitation, road widening and brownfield development.

If any utility services are going to be potentially affected by construction the designer may choose to arrange for utility pilot trenching over critical services as highlighted on the base plan and agreed with the Auckland Transport representative. Liaison and necessary permits are to be obtained as required, for example, Road Opening Notice / Corridor Access Request (CAR) and Utilities Approval.

For pavement rehabilitation, a Ground Penetrating Radar (GPR) survey instead of intrusive excavation with pilot trenching in the road carriageway should be considered for preliminary identification and location of underground services. The information and outputs regarding the location of utility services is to be included with the design report.

2.6 SUBSOIL DRAINS

Subsoil drainage is one of the most important requirements of pavement construction and to be normally installed along the edges of all road pavements, where low points are created with channel drains.

This section sets out the material and construction requirements for the subsoil drainage which shall be provided on both sides of all road pavements beneath the kerb and channel or edge beams, beneath permeable central median areas and anywhere where sub-surface water is likely to collect in the road pavement.

All materials and construction shall comply with the relevant requirements of the following standard specifications together with the further provisions herein.

- AT TDM Kerb Design drawing including Standard Detail KC0007
- NZTA F/2 (2013) Pipe Subsoil Drain Construction
- TNZ F/7 (2003) Geotextiles
- NZS 4402 (1986) Methods of testing soils for civil engineering purposes

The filter material shall comply with all aspects of TNZ F/2 and shall be clean, durable stone and have a crushing resistance of not less than 100kN when tested with NZS 3111.

Alternative drainage materials such as crushed rock such as D7/D20, without fine aggregate, and encased in a Class 3 filtration geotextile may be acceptable in exceptional cases for rural roads, however this needs to be approved through the EA process. Perforated pipe is to be a road subsoil drainage type system complying with TNZ F/2 with a pre-fitted filter sock.

For existing roads where subsoil drainage is to be installed in the carriageway for other technical reasons, it is acceptable to use 12mm no-fines concrete with 28 days crushing strength of minimum 20MPa, for backfill, and encased in a Class 3 filtration geotextile. This, however, is to be discussed and agreed with Auckland Transport prior to undertaking any site work.

Subsoil drains shall be designed to connect to catchpits with the subsoil pipe invert level above the soffit level of the catchpit outlet pipe at the outflow end. The uphill end of the subsoil pipe shall be capped and either start at a capped flushing eye in a surface box or connect to an upstream catchpit. Subsoils shall be laid at a continuous grade without depressions that could pond water and shall discharge into catchpits at spacings not exceeding 100m. Where there are no catchpits, subsoils may discharge into stormwater manholes in a location and at a level approved in the EPA process.

2.7 PAVEMENT MODELS

All modelled mechanistic /empirical designs must include summary output sheets of the mechanistic modelling from either CIRCLY or AUSTPADS. CIRCLY 7 is the preferred version for modelling, however, other versions of CIRCLY can be used provided the traffic is characterised appropriately. The output files showing parameters and pavement configuration used to model deflection and curvature function should also be provided.

AT accepts design outputs for the pavement profile from either CIRCLY or AUSPADS analyses. If the geotechnical investigation proves that the subgrade improves with the depth below finished subgrade, then interpretation of actual load depth influences up to 2m for subgrade. However, current practice is to assume the subgrade depth to semi-infinity unless there has been sound geotechnical support to better subgrade layer within 2m below the finished subgrade level.

Mechanistic modelling of pavement thickness shall not be used for subgrade rutting failure of unbound granular pavements. Figure 8.4 in Austroads Guide to Pavement Technology Part 2 (AGPT-2) latest version shall be used. However, mechanistic modelling shall be used to check and demonstrate central deflection and deflection curvature compliance on top of the basecourse layer as provided in Table 25: Maximum Benkelman Beam Deflections on top of Basecourse (prior to Surfacing).

Mechanistic pavement modelling or compliance checking of deflection and curvature is not required for Catalogue Design options.

2.8 SELECTION OF PAVEMENT TYPE

2.8.1 Flexible Pavement

Flexible pavements include unbound and modified granular basecourse or subbase course, foamed bitumen basecourse, cement bound basecourse or subbase, and structural asphalt, all with thin asphalt or sprayed chip seal surfacing.

AT road networks comprise largely flexible pavement, both sealed and unsealed. There has been some concrete pavement built in the AT network in the early 1900s, and this is currently overlaid with thin asphalt and/or sprayed chip seal surfacing.

Generally, it is recommended to adopt flexible or rigid/flexible composite pavements in new design, partly because of the high capital cost of construction and difficulty with repairs and maintaining skid resistance of rigid pavements.

2.8.2 Rigid Concrete Pavement

Rigid pavement is relatively inert to chemical attack and far less susceptible to surface distress than asphalt or chip sealed road pavement materials. Therefore, it is considered as an appropriate option in roads with frequent stationary or static parking usage by heavy vehicles, such as public transport bus terminals and interchanges, side streets with regular loading/unloading activities in industrial areas, vehicle crossings, raised safety platforms, roundabouts and raised or at-grade intersections, and traffic calming raised platforms / pedestrian crossings.

Rigid pavements include conventional jointed reinforced concrete, steel or synthetic fibre reinforced concrete and roller compacted concrete.

AT has adopted rigid pavement on almost all footpath and off-road cycleways to improve the overall durability and minimize the maintenance needs. Elastic modulus and shear modulus of concrete are much greater than those of asphalt mixtures. TDM standard drawings are available for many of the rigid pavement components commonly used in the Auckland region including in new subdivision road construction.

Construction of carriageway using rigid pavement may be considered by pavement designers when the pavement is anticipated to have movement of traffic less than 40kph in normal circumstances and when there are no requirements for utility road opening works along the carriageway section for roads or major intersections and roundabouts. This also applies to bus terminals and stations where static bus loadings are expected.

Rigid pavement can also be used to create a subbase course that spans weak subgrade material in situations where existing utilities would otherwise require relocation for the pavement design. The concrete subbase course is laid on a Class C geotextile to prevent subgrade fines migration upward. This is an example of a composite rigid/flexible pavement.

Concrete is a mixture of fine and coarse aggregate, water, cementitious binder, and admixtures. It is used on pavements in several ways. Principal pavement types are as follows:

1) Plain concrete pavement which is unreinforced.
2) Jointed reinforced concrete pavement, which is typically mesh reinforced, with square dowelled joints at spacings of 8–12 m. The longitudinal reinforcement does not cross the transverse joints. continuously reinforced concrete pavement (Figure 2.13) A concrete pavement containing relatively heavy longitudinal reinforcement and having no transverse joints.
3) Fibre reinforced concrete pavement, generally reinforced with steel fibres.
4) Concrete segmental pavement consisting of a surfacing of interlocking precast concrete pavers.

Concrete pavement has a relatively high embodied carbon footprint. However, recently lower carbon concrete options are being produced and used for some industrial pavements and traffic calming assets. As technology advances rigid or composite rigid/flexible pavements may become more commonplace in roads.

2.8.3 Pavers / Paving Stone

Pavers, or paving stones, are flat outdoor flooring materials, a segmental pavement used for hardscaping projects especially for pedestrian area, vehicle crossing, and limited in traffic areas primarily intended for delineating and aesthetic purpose. Pavers can be made from a wide range of materials, including concrete, stone, or clay, and are available in different shapes, sizes, colours, and styles.

Basalt and granite are igneous rocks often used for footpaths, limited section of the road pavement with concrete basecourse. It's also used for masonry projects. A separate practice note will be developed on the use of paver as pavement for various purposes. All supplied pavers shall have recently tested product certificates conducted in IANZ accredited laboratory as a minimum requirement for breaking strength, modulus of rupture / flexural strength, abrasion resistance, slip and skid resistance at wet condition to the site-specific geometrical condition and site profile, and water absorption as per the details provided in EN1341:2012 or similar approved standards including ASTM. Permissible tolerance in thickness should not be more than 3mm.

For general guidelines, Class 4 pavers for public pedestrian areas - footpath and cycleway areas, marketplaces occasionally used by delivery vehicles and emergency vehicles and Class 6 paver for all vehicle traffic areas including carparking with minimum breaking strength of 9kN and 25kN respectively will be required.

The designer shall submit a bespoke design with all test certificates for using pavers and paving stone for vehicle traffic areas and obtain AT approval for departure from standards. It is not recommended to use such paving stone on major bus routes.

2.9 PAVEMENT TYPE SELECTION

Suitable pavement design types for the movement significance as defined in the Roads and Streets Framework (Auckland Transport 2020) are listed in Table 6: Suitable Pavement Designs for Various Traffic Levels below. Urban networks are considered to have speed limits less than 60 kph, while rural networks have speed limits greater than or equal to 60 kph. This table indicates the types of pavements that are likely to perform well in these traffic environments, however, other factors, such as sustainability, a circular economy, construction efficiencies and safety in design (for example maintenance considerations) may influence the type of construction adopted.

Unbound or modified granular pavements are not considered suitable designs for M3 classifications - primary and arterial roads. The use of unbound or modified granular pavements in M2 road classifications requires a risk assessment for the site to determine if a lower risk treatment might be more appropriate. Such a risk assessment would involve looking at the types of traffic loading that might be expected and considering if the heavy vehicles are likely to be turning frequently or driving straight along the alignment.

Table 6: Suitable Pavement Designs for Various Traffic Levels

Movement Classification	Indicative Design Traffic	Suitable Pavement Treatments	
		Urban (</= 60 kph)	Rural (>60 kph)
Higher volume M3 (Primary / Arterial Collector roads including Rapid Transit Routes, major bus and HPMV routes)	Greater than 5×10^6 ESA	▪ Structural asphalt with unbound or modified granular subbase or lean mix concrete subbase. ▪ Foamed bitumen stabilised pavement with thin asphalt surfacing	▪ Structural asphalt pavement ▪ Foamed bitumen stabilised pavement ▪ Cement bound subbase with unbound or slightly cement modified basecourse up to 1.5% cement
Medium volume M2/M3 (Secondary / District Collector with bus routes)	From 1×10^6 to 5×10^6 ESA	▪ Structural asphalt pavement ▪ Foamed bitumen stabilised pavement ▪ Cement bound subbase with unbound basecourse	▪ Foamed bitumen stabilised pavement ▪ Cement bound subbase with unbound basecourse. ▪ Slightly cement modified basecourse up to 1.5% only
Lower volume M2 / M1 (Secondary / District Collector, Industrial / Commercial Roads or Service lanes including all roads with bus routes)	From 1×10^5 to 1×10^6 ESA	▪ Structural asphalt pavement ▪ Foamed bitumen stabilised pavement ▪ Cement bound subbase with unbound basecourse. ▪ Cement modified basecourse ▪ Unbound granular pavement	▪ Foamed bitumen stabilised pavement ▪ Cement bound subbase with unbound basecourse. ▪ Cement modified basecourse ▪ Unbound granular pavement.
Lowest volume M1 (Local Roads, Residential Street, Accessways and off-road including carpark)	Less than 1×10^5 ESA	▪ Two coat chip seal surfacing or AC wearing course ▪ Unbound granular pavement with unbound premium basecourse and unbound subbase.	▪ Two coat chip seal surfacing or AC wearing course ▪ Unbound granular pavement with unbound basecourse, unbound subbase.
Roundabouts and Approaches (	HCV >= 100 per lane per day	▪ Structural asphalt concrete pavement ▪ Rigid concrete pavement	▪ Structural asphalt concrete pavement ▪ Rigid concrete pavement
Signalised Intersections and Approaches Refer Section Design traffic (For all Road Classifications)	HCV >= 500 per lane per day	▪ Structural asphalt concrete pavement ▪ Rigid concrete pavement	▪ Structural asphalt concrete pavement ▪ Rigid concrete pavement

2.9.1 Basecourse on Stabilised Subbase Course

Where a stabilised subbase course underlying an unbound or modified basecourse is part of the pavement structure, the pavement design report shall detail how the basecourse shall be effectively drained.

If a kerb and channel or kerb and nib is present, the minimum thickness of AP40 unbound granular basecourse under the kerb and channel is 50mm and it must drain to the pavement drainage system to discharge sub-surface water. Additionally, the subbase course stabilisation must terminate at the lip of proposed concrete channel to allow moisture within the basecourse layer to reach the subsoil drainage.

Regardless of whether it is unbound granular or modified / stabilised subbase, the overlaying granular basecourse must be a minimum 130mm thick and meet the basecourse thickness allowing tolerance during construction requirements of Figure 8.4 from Austroads AGPT-2, whichever is greater.

2.9.2 Surfacing

The types of surfacing will be determined by the traffic levels and the stress environment. Indicative surfacing types are presented in Section 7.0, irrespective of the movement significance. For all roads in the new sub-divisions in urban environment are to have asphalt concrete finished surfacing on membrane seal.

The chipseal may not be suitable where the annual average daily traffic on both directions is greater than 10,000 vehicles per day. The risk of construction traffic damaging chipseal should also be considered when selecting a wearing course. The use of asphalt resurfacing generally relates to main routes in the network with an AADT of more than 10,000 vehicle per day, although there may be exceptions to this. The curvature function should be a key design consideration for all roads where a thin asphaltic concrete (AC) surface is to be used.

Until any update is undertake, decision on resurfacing on existing road and seal extension to existing metal to be followed as per Auckland Transport Guidelines – Reseal Guidelines Reseal-Guidelines (Feb 2014) and Seal Extension Guidelines Seal-Extension-Guidelines (Feb 2014) respectively provided in the Appendix F.

For resurfacing on the existing pavement using asphalt concrete for traffic levels greater than 10MESA, it is advised to check the curvature function of the existing road pavement for asphalt surfacing on existing road a pavement curvature of less than 0.15 mm FWD curvature, D_0 -D_{200} is expected to achieve 10 years or more surfacing life. It is necessary to identify and undertake pavement repairs before asphalt concrete resurfacing.

Detailed guidelines on the asphalt surfacing is provided in NZTA Asphalt Surfacing Treatment Guidelines 2012 version 2.1 https://www.nzta.govt.nz/assets/resources/asphalt-surfacing-treatment-selection/docs/asphalt-surfacing-treatment-selection-guidelines.pdf.

Table 7: Surfacing types for the Movement Significance

Movement significance	Suitable Pavement Surfacing	Surface Types
M3 (Arterial and Primary Collector roads including Rapid Transit Routes, major bus and HPMV routes)	Dense graded asphalt (NZTA M10) or stone mastic asphalt (NZTA M27) on Grade 4 membrane seal, or a two-coat chip seal if it is to be trafficked prior to paving – this applies to both rural and urban roads.	AC14, AC10, or SMA10 to meet NZTA T/10 skid resistance requirements for micro and macro texture.
M2/M1 (Secondary Collector including local roads with minor bus routes and local residential streets)	Urban - Dense graded asphalt (NZTA M10) on a membrane seal, or a two-coat chip seal if it is to be trafficked prior to paving in all new roads.	Urban - AC10, DG10
	Rural - Chipseal (NZTA P3 or P4) – on rural road or seal extension.	Rural - Grade 3/5 two-coat chip sealing and Grade 4 for resealing.

Movement significance	Suitable Pavement Surfacing	Surface Types
Roundabout & Intersection	Dense graded asphalt (NZTA M10) or stone mastic asphalt (NZTA M27) on a membrane seal. If road to be trafficked before asphalt surfacing provide a two-coat chip seal unless it is rigid pavement (concrete surfacing).	AC14 or SMA10 when required.
Carpark / Footpath or Shared Path (Cycle path)	Dense graded asphalt (NZTA M10) on a membrane seal.	DG7/ DG10 on grade 5 membrane seal
Bridge Deck	Dense graded asphalt (NZTA M10) or stone mastic asphalt (NZTA M27) on Grade 4 membrane seal; this applies to both rural and urban roads.	AC14, AC10, or SMA10 using polymer modified binder as per NZTA T/10 skid resistance requirements for texture.
Levelling-mix	Asphalt concrete thin surface to be used for designed level such over bridge decking and road pavement following membrane sealing	All mixes including DG7, AC7, AC10. DG10 and AC14.
Membrane seal	Grade 4 or 5 chips sprayed on approved bitumen binder (without cutter) – preferably rapid breaking industry approved emulsion binder	Minimum residual application rate 1.4litre/m2 for granular basecourse and 1litre/m2 minimum for resurfacing on existing surface on milled asphalt surface.
high stress sites, such as turning circles, cul-de-sacs > 100HCV per day	Dense grade asphalt as a wearing course	AC10 or AC14 surface (PG64 V or E performing grade binder depending on the traffic loading.

Stone mastic asphalt may be considered where a better performing skid resistance and a low noise surfacing is required.

Membrane seals should not be trafficked prior to paving the overlaying asphalt except by construction traffic necessary for the paving operation. If there is a likelihood of the membrane being trafficked, then it is recommended to a two coat or racked in chipseal using Grades 3 and 5 chip. The NZ Chip Sealing Design Manual is used to determine the application rate, with a minimum 1.8litre/m^2 to be used for two coat chip sealing. The chipseal should be trafficked for a minimum of one week, and preferably four weeks, prior to application of a tack coat and the overlaying asphalt wearing course. Loose chips are to be fully swept from the surface prior to asphalt paving.

2.9.3 Bridge Deck Surfacing

Bridge decks typically have an asphalt wearing course over an asphalt levelling course, to accommodate minor shape correction during construction, provide water proofing, and allow for future surfacing replacement without milling the deck of the bridge. The following minimum requirements apply to AT bridge deck asphalt surfacing:

- The bridge deck wearing course mix will typically match the wearing course on the pavements approaching the bridge and may therefore be either dense graded asphalt AC14 or SMA10. It is required to have bridge approaches at least 60m in both directions to have an asphalt type of surfacing. Minimum thickness to be 55mm for AC14 or 40mm for AC10 and SMA10 surfacing. It is not recommended to use DG10 over the bridge decking as final surfacing unless it is on a local road without bus services.
- Binder selection for bridge surfacing shall use polymer modified binder in accordance with NZTA M10 and M01-A specifications to mitigate potential reflective cracking from the concrete decking. Although

binder grade is dependent on the level of the traffic, it is preferable to use heavy, PG64 H for local roads or very heavy, PG64 V grade performing grade binder for major roads as provided in NZTA M01-A.

- Designers shall check with the bridge design or maintenance engineers whether the total thickness of both levelling and surfacing asphalt can be tolerated by the bridge structure.
- The levelling mix layer shall be made of DG7 or AC7 for up to 30mm and DG10 or AC10 mix for up to 60mm. The minimum thickness of the levelling course shall be 25mm to provide protection to the bridge deck during surfacing replacement.
- A tack coat is to be applied to the concrete surface typically up to 0.5litre/m2 residual binder depending on the surface texture and the finish, ideally confirmed by trial during construction. The tack coat may be a specialised proprietary product in which case it must be applied strictly as per the manufacturer's instructions.
- The dead load from levelling and wearing course asphalt concrete shall be included in the structural design of the bridge.
- Consider providing expansion / contraction joint between the abutment and road approaches minimum 10mm saw cutting to the adjoining assets – barriers, footpath, kerb and channel and road pavement and fill with Fosroc Nitoseal SC820 or similar sealant to accommodate movements of the bridge deck and mitigate damage to the assets

2.10 TRAFFIC LEVELS CONSIDERATION

It is advised to note that the design traffic for the various classifications is estimated because the Roads and Streets Framework does not explicitly have traffic volumes ranges. In addition, the Roads and Streets Framework develops the boundaries of these ranges may not be correct. Notwithstanding this, the treatments will remain appropriate for the indicative design traffic levels, but the risk approach should be considered on a case-by-case basis and evaluated as being appropriate for the movement classifications.

While the indicative design traffic levels are expressed as equivalent standard axles (ESA), all designs that involve bound layers should be designed according to Austroads AGPT-2, developing a representative load distribution for the heavy vehicle axles, before calculating the ESA level.

Designers should consider the effect of vertical stresses at the pavement surface, and the impact on the tensile strains at the bottom of bound layers (asphalt or cemented materials) and compressive strains at the subgrade level. Where thin asphalt is being considered in areas of high shear stress, the Austroads pavement design guide provides the following advice:

- During certain manoeuvres, such as braking, turning, and travelling uphill, heavy vehicles apply horizontal loads to the pavement, which are currently not considered in the design model. For thin surfaced granular pavements, the stresses generated by these loads are concentrated in the upper pavement layers and can have a significant detrimental impact on the performance of the surfacing.
- Traffic multipliers for intersections, approaches to intersections and roundabouts, and roundabouts shall be used and applied to the calculated N_{DT} or ESA value. Refer to Section 4.0 for details.
- Caution is advised in adopting the thin asphalt surfaced pavement option on roundabouts and intersections because the dominant damage mechanisms are not considered in the design methodology, which assumes sufficient strength of the materials. In both rural and urban situations, the designer should consider the requirements of NZTA's Technical Advice Note 17/01 which requires structural asphalt for roads with high volume heavy vehicles roundabouts and intersections. Refer to Section 4.0 for details.

In some circumstances, more structurally robust designs may be required for a road category due to site constraints, such as shallow services or to address uncertainty in loading assumptions. The designer should clearly discuss these scenarios in the pavement design report.

2.11 CONSTRUCTION TRAFFIC DEFLECTIONS

Expected construction traffic and deflection on the finished basecourse (layer below the sprayed chip seal or thin asphalt surfacing) shall be documented in the pavement design report. Acceptance criteria shall be shown on the drawings for confirmation prior to placing surfacing on basecourse. These deflection requirements will also be detailed as hold points in the inspection and test plan.

Details on the deflection requirements are provided in Table 25: Maximum Benkelman Beam Deflections on top of Basecourse (prior to Surfacing).

2.12 JOINT CONSTRUCTION

Joint locations and construction details such as stepping, particularly relating to road widenings shall be addressed in the design with appropriate treatments recommended. Longitudinal pavement joints, where unavoidable, should be positioned under lane edges or centre of the lanes. The minimum width for any road widening construction is to be not less than 1000mm for major arterials and 500mm for collector and local roads to enable compaction of the pavement materials. Minimum lap distance for each layer relative to the layer below joint shall be as follows:

- For longitudinal joints - 300mm.
- For transverse joints for - 500mm minimum for local roads and 1000mm minimum for primary collector and arterial roads with or without bus routes.

Note that a 3m width is the minimum for pavers laying structural asphalt pavements and 2.5m width is the minimum width for in-situ stabilisation equipment and suitable compaction equipment.

Cold joints may require removal of existing layer by saw cutting at least 150mm or providing special treatment approved by Auckland Transport to prevent any opened joint.

For joints between new and existing asphalt the joint shall be sawcut, the vertical face of the sawcut shall be tack coated with bitumen and the finished joint shall be sealed with a proprietary surface bandage such as Polyflex Crafco 2 or equivalent approved bitumen sealant.

3.0 INVESTIGATIONS

In a Greenfield site for new development work, geotechnical investigations are to be carried out and an interpretative report is to be provided with a presumptive pavement design subgrade soaked CBR (California Bearing Ratio). The level of investigations should follow the recommendations detailed in the NZ Guide to Pavement Evaluation and Treatment Design. Given the potential level changes in greenfield projects designers should ensure that the subgrade is investigated at and below the proposed road formation levels. Where available, wider project level geotechnical investigations should be analysed to assist in determining the design parameters.

Similarly, for carriageway widening, rehabilitation, or strengthening of existing pavements the level of investigations should follow the recommendations detailed in the latest version of NZ Guide to Pavement Evaluation and Treatment Design.

3.1 FIELD INVESTIGATION

3.1.1 Falling Weight Deflectometer Test

For pavement rehabilitation design or road widening of the existing roads, it is recommended to carry out Falling Weight Deflectometer (FWD) investigations at 20m centres staggered for tests in both directions. Designers are reminded that FWD results are analysed using isotropic material models and the calculated material parameters are not suitable for direct input into pavement models.

3.1.2 Dynamic Cone (Scala) Penetration Test

For road widening and pavement rehabilitation, test pits or hand auger boreholes including dynamic cone (Scala) penetrometer inferred CBR or Field Shear Vane testing may be carried out to assess the subgrade strength for pavement design. Generally, natural subgrade in the Auckland region has been recorded in between 2% and 3.5% and subgrade strength in Takanini / Papakura area is much lower or CBR less than 1 due to presence of underlying peat soil. Indicative soil map for this area is provided in Appendix D.

Subgrade testing shall be taken at the design subgrade level, and Scala testing and hand augers should be undertaken to a minimum depth of 1.5m below design subgrade level.

3.1.3 Exploratory Test Pits

Test pit investigations will involve a description of the subgrade soil, Scala penetrometer and shear vane tests every 100 metres with a minimum of three test pits per treatment length. Subgrade shall be sampled in the locations of the Scala testing and be tested to determine the soaked CBR, the unsoaked CBR and the moisture content of the in-situ soil. Unsoaked CBR results allow better correlation with CBR inferred from Scala results, however, these tests may be removed if a good correlation is already available due to in-situ moisture contents being high. If the subgrade consists of a granular material, FWD or Benkelman Beam testing may be a more appropriate method to estimate the subgrade strength.

Brownfield pavement investigations shall follow the recommendations of the latest version of the NZ Guide to Pavement Evaluation and Treatment Design. The test pit size may be reduced provided the subgrade can be reached and logging of the pavement layers, including photos, can be achieved. Should the homogeneity of the site be demonstrated the test frequencies might be reduced using a risk-based approach however this would require approval from Auckland Transport's Chief Engineer. If the proposed road usage is changing from the traditional usage, for example including a bus route, then the existing pavement needs to be carefully investigated to ensure it has the structural capacity for the future traffic.

All intrusive ground investigation logging, sampling, and testing shall be performed by Laboratories with the appropriate IANZ accreditation.

3.2 LABORATORY TESTING

Laboratory testing is required to properly characterise the pavement materials and provide design parameters.

All Laboratory Testing is to be carried out by an approved IANZ (International Accredited New Zealand) certified Civil Engineering Laboratory.

Sampling and testing scope is the responsibility of the designer to ensure the quantities and testing are sufficient to establish the condition and properties of the various existing subgrade and pavement layers materials, and to support and justify any design properties chosen.

The designer shall recommend sufficient testing to determine material properties of the existing subgrade and pavement including:

- **Soaked CBR Test of Natural Subgrade** - Remoulded soaked CBR testing of the subgrade shall be carried out. Test and report to NZ Standards. Compaction shall follow the NZ Standard 4402 Test 4.1.1. The swell value should also be reported.
- **Modified Soaked CBR** - A modified soaked CBR using stabilising agents, (for example, lime, cement, or slag (Klockner Oxygen Blowing Maximillianshette- KOBM) or blend of these) shall be carried out for subgrade stabilisation in accordance with NZTA B9 pilot specification and NZS 4402.
- **Particle Size Distribution (Wet) Test** – grading of the existing basecourse and/or subbase course shall be determined for materials to remain in place or to be modified or stabilised. Test and report to NZ Standards, NZS 4407 Test 3.8.
- **Unconfined Compressive Strength (UCS) test** – soaked or dry UCS using stabilising agents, (i.e. lime, cement, KOBM or blend of these) shall be carried out for existing aggregates to be modified or bound. Testing shall be in accordance with NZS 4402 Tests 4.1.1 and 6.3.1.
- I**ndirect Tensile Strength (ITS) test of modified granular materials** – soaked and/or dry ITS shall be obtained using stabilising agents, (i.e. lime, cement, KOBM, emulsion and foamed bitumen) according to NZTA B5 specification and NZTA T19.
- **Atterberg Limit** – plasticity index, plastic limit and liquid limit must be determined in accordance with relevant New Zealand standards.

The outputs of all laboratory test results are to be summarised in the design report and included in full in the appendices. This includes existing pavement materials where road widening, or pavement rehabilitation activities are to be carried out.

For new greenfield developments, the source of imported materials may not be known at the time of design. In that case investigation and testing shall be on known existing materials at cut level or being used for filling to determine the appropriate soaked CBR for design.

If sources of imported materials are known, then laboratory testing results from quarries, or new testing, is highly recommended.

3.3 SAMPLE MASS OF SOIL FOR TESTING

The total mass and number of representative samples required depends on the soil group and the tests to be carried out for carrying out laboratory testing for moisture content, specific gravity, particle size distribution, clay and plasticity indices, compaction (Proctor) and California bearing ratio etc. In general, it is recommended to take a minimum of 3 samples, and the total mass should not be less than the following:

- Fine-grained soils 500 g

- Medium-grained soils 5 kg
- Coarse-grained soils 30 kg

The actual mass of sample required can be assessed by multiplying the mass given in the table below by the maximum number of tests envisaged. Sample number, sizes, and frequency of relevant laboratory and field testing for the various materials can be referred to NZ Guide to Pavement Evaluation and Treatment Design.

3.4 CALIFORNIA BEARING RATIO (CBR)

3.4.1 Subgrade Strength Measurement by Field Testing

Auckland natural subgrades typically have subgrade strength measured in California Bearing Ratios (CBRs) in the range of 2% to 3.5%. The Dynamic Cone Penetrometer (DCP) is used to determine underlying fine grained soils strength by measuring the penetration of the device into the soil after each hammer blow. A two-person crew operates the DCP and records data manually[1]. For most soil types, best correlation with CBR is achieved when the Scala DCP mm/blow is calculated from a weighted average of blows/50 mm for the first three 50 mm intervals using weightings of 0.7, 0.2 and 0.1 for each interval. It is important to emphasise that the DCP test permits determining the in situ CBR value of especially fine-grained soils with any range of strength but to a limited depth ranging from 80cm to 150cm depending on the type of soils.

DCP testing shall comply to NATA- Accredited DCP testing based on AS 1289.6.2.3.2: Soil and Strength and Consolidation Tests. This is the method of determining the penetration resistance of soil with a 9kg dynamic cone penetrometer. The conversion of test data to CBR values is based on A.J Scala: Simple Methods of Flexible Pavements Design Using Cone Penetrometers; Proceedings Second Australia-New Zealand Conference Soil Mechanics and Foundation Engineering, Christchurch, N.Z, January 1956.

Several relationships between CBR and penetration (in mm/blow) have been reported in the literature, e.g. Mulholland (1984), Schofield (1986) and Smith and Pratt (1983). When using the cone penetrometer extensively for subgrade investigation, other CBR testing alternatives should be used to confirm the validity of the CBR/penetration relationship adopted as provided in the Austroads AGPT-2 Figure 5.3 Correlation between DCP and CBR for fine grained cohesive soils included in the Appendix C.

DCP reading in the summer months may not be well represented the actual subgrade CBR because such tested or measured values will likely provide higher readings than test results measured in winter months with saturated soil condition. It is also advised to carry out laboratory testing 4-7day soaked samples to determine the representative of subgrade strength to confirm with target CBR in the design. Cautions are to be taken inferring a direct reading from the laboratory as some types of remoulded samples of soil at field moisture content and density give CBR values, which are generally higher than the in-situ CBR test results using DCP. These findings are based on field Study carried out and published in Australian Road Research 13(4), December 1983, by R.B. Smith and D.N. Pratt - A Field Study of In Situ California Bearing Ratio and Dynamic Cone Penetrometer Testing for Road Subgrade Investigations.

Sometimes a CBR of less than 2% is encountered, particularly in areas with peat soils, or in low lying wet soils and remoulded weak soils. Peat soil with CBR <1% can also be found in South Auckland suburbs to Papakura and Takanini area. CBR values up to 5% are occasionally encountered including on engineered fill, and some volcanic tuff, but this is infrequent.

3.4.2 Subgrade CBR by Laboratory Testing

The laboratory test results should be analysed alongside the in-situ subgrade testing and be considered against the presumptive values (for example see the values provided in Austroads AGPT-2 section 5.7 for the materials

[1] One Scala blow per 50 mm can be inferred to be CBR 3.5 or 3.7%, for the purpose of the TDM it is assumed that one blow per 50 mm is equivalent to 3.5%. Similarly, one blow per 100 mm is assumed equivalent to a CBR of 2% and one blow per 200mm is equivalent to a CBR of 1%. Refer Appendix C.

observed. The adopted subgrade CBR should consider the site location, groundwater levels, and the current environment.

A soaked CBR test swell value of 2.5% or greater indicates the subgrade material is expansive. An example of such material is the Northern Allochthon (Onerahi Chaos) material found between Albany and Orewa. Expansive materials can create shape problems in pavements and berm areas by expanding when moisture contents increase and contracting when moisture content decrease. It is prudent to maintain an equilibrium water content in expansive subgrade material. Lime stabilisation of the top 300mm of expansive material and locating pavement drainage within the stabilised layer (shallow the depth of subsoil trenches) will help mitigate risk of wetting and/or drying of the expansive material.

In selecting a design CBR a subgrade under an existing pavement might adopt an unsoaked subgrade CBR, rather than the soaked value, if the pavement has existing subsoil drainage in good working condition and this has been proven for design with CCTV imagery in the existing subsoil pipes. In contrast, the laboratory soaked CBR is more appropriate for a greenfield pavement.

3.4.3 Subgrade Sensitivity

Subgrade sensitivity is a construction risk that should be carefully considered, the approach detailed in the latest version New Zealand Guide to Pavement Evaluation and Treatment Design Section 4.5 should be followed, field shear vanes peak and remoulded strength ratio and soaked CBR tests should be used to check the sensitivity of the subgrade. Soils with a shear strength ratio of 4 or more should be considered as likely to be weakened during construction due to construction loading on subgrade areas. Adoption of an appropriate design soaked CBR to allow for this potential weakness should be undertaken. This is particularly important in narrow corridors, such as widening of existing pavements where construction activity has no choice except to traffic over subgrade materials.

For pavement designs, adopting a design CBR greater than 3.5% will not be generally acceptable without significant evidence of investigation and testing by geotechnical engineer of in situ subgrade demonstrating that a higher soaked CBR value is justified by carrying out laboratory testing of soaked CBR of at least three representative samples.

3.4.4 Subgrade CBR Values for Catalogue Design

The catalogue designs have been developed assuming design CBR values of 2% (with a contingency design for CBR 1%), 3.5% and 5%. Subgrade improvement layers adopted for the catalogue designs are shown in Table 5.

Table 8 Materials Used for Subgrade Improvement Layers

Description	Subgrade Strength - CBR (%)		
	2 or less	2 to 3	> 3.5
Recommended Type of Subgrade Improvements	Black sand encased in Class C geotextile	Chemical stabilisation using lime and cement, or black sand encased in Class C geotextile	Class C geotextile separation layer

Note that in-situ chemical stabilisation may not be possible on very sensitive soils in the CBR 2 to 3% range as the stabiliser weight may shear the subgrade below further weakening it. A shear vane sensitivity test shall be carried out to determine sensitivity.

3.4.5 Subgrade Improvement Layers (SIL)

Weak and sensitive subgrade identified in many parts of Auckland is the main problem during the construction of pavements for roading projects. This section provides on subgrade improvement layers to improve the properties and increase the strength of weak, sensitive, swelling soil or highly saturated peat soil for use as a subgrade for pavement structural sections. Modelling of the pavement shall show both a subgrade improvement layer (SIL) and the existing subgrade underlying the SIL.

SIL depths determined in bespoke designs, over weak subgrade, to satisfy subgrade rutting criteria are unlikely to be sufficient to meet the pavement deflection requirements that are detailed in Section 8.8.2 Austroads, 2017. Additional SIL and/or pavement material depths are likely to be required.

The ability to constructed subgrade improvement layers for existing roads requiring widening or pavement rehabilitation may be limited by the presence of underground utilities. If this is the case, then the catalogue designs are not appropriate, and alternative treatments should be detailed in a bespoke pavement design approach.

Geogrids, a synthetic commonly known as polypropylene or polyester or glass fibres, membrane made is stiff and rigid or flexible depending on the polymer and manufacture process. Traditionally, geogrid and geotextiles are used to improve soil and subgrade improvement and as a reinforcement and/or stabilisation of natural soils and aggregates obtain better resistance to the vehicle loadings including improve compaction.

However, we consider that such provision can be used for only for constructability including protection of pavement layers for delivery of construction materials while the carriageway is being used for haulage. Geogrid normally is expected to help in keeping granular basecouse or subbase interlocked providing rutting resistance on vertical loading and as well as protecting the subgrade during construction.

Generally, the following types of subgrade improvements are commonly used in the pavement design:

1) Imported or site won, engineered fill soil,
2) Cut and fill with granular base materials (mechanical stabilisation),
3) Chemically stabilised subgrade and,
4) Black sand wrapped with Class C geotextile.

3.4.5.1 Imported or Site Won Engineered Fill

When constructing a SIL, the engineered fill material shall have a minimum laboratory soaked CBR of at least 8% and minimum field shear vane strength of 140kPa. Such engineered fill material can then be used for subgrade improvement and pavement design using a CBR of 50% of the laboratory soaked CBR. However, Austroads rules (Section 8.2.2 Austroads, 2017) limit the strength gain that can be achieved above underlying layers (strength doubles every 150mm of lift of SIL), and this may be less than the strength the SIL material can achieve.

3.4.5.2 Stabilised Subgrade using Additives - Lime and or Cement

Lime is most used with highly plastic soils and cement is used with less plastic soils (plasticity index less than 10) and emulsified asphalt can be used with sandy soils. While calcium hydroxide (hydrated lime) is the active compound in the stabilisation reaction, calcium oxide (quicklime) is generally used for pavement construction. Quicklime is converted to hydrated lime in-situ by adding water, a process called slaking. Emulsified binder is less common in this region and can only be used as per industry approved specifications and stabilisation methodology.

As in engineered fill, the existing subgrade will limit the achieved strength of the improved subgrade as stated in the Austroads. Additionally, the strength achieved will be less than that achieved in the laboratory mix design due to difference between compaction in a mould in the laboratory and field compaction. The maximum design subgrade CBR for design should not be more than 50% of the laboratory soaked CBR% for chemically treated soil under reactivity testing. The maximum assigned design value is CBR 10% and that would require a laboratory reactivity testing for soaked CBR of 20% or more.

For cement and lime stabilisation the depth of effective subgrade shall be modelled as 25mm less than the actual stabilised depth. For example, if the stabilisation depth is 250mm during construction, then the depth of effective stabilisation is 225mm in the pavement design model. In addition, it is important to note that the maximum depth of in-situ stabilisation of a subgrade is 300mm in the current industry and in that case the effective stabilisation for pavement design is 275mm. Therefore, the construction drawing shall show the stabilised depth to be 300mm.

3.4.5.3 Black Sand with Class C Geotextile

Woodhill sand or black fine-grained sand can be used for organic or peat soil or other weak soils where subgrade CBR ranges from 1.5% to less than 1%. Following cut to level to a designed cross-fall, a Class C geotextile fabric shall be laid to contain and fully encase sand to prevent loss of material due to sub-surface drainage. Minimum thickness of the sand layer shall be 300mm and it should be track rolled only to compact it. For peat soil, this can be extended 500mm to 700mm depending on the saturated condition of the underlying peat soil. Indicative locations of the peat soil are provided in the Auckland Region Map in the Appendices of the TDM.

In addition, Experimental results show that when sand and cement is added, the hardness and load-bearing capacity of the soft ground increase significantly, particularly if the underlying soil is in a highly saturated condition with moisture content more than 50%.

3.4.6 Bespoke Design Subgrade CBR Values

For bespoke pavement designs, adopting a design soaked CBR greater than 3.5% for a natural soil will not be acceptable without significant evidence of investigation and testing of subgrade demonstrating that a higher soaked CBR value is justified for the design. Subgrade testing using DCP in dry summer season could likely to provide a higher reading, which should be verified with laboratory testing to at least 3 representative samples. However, DCP testing in winter weather in saturated soil condition may be considered as a basis of the design taking the 10% of all tested results.

SIL depths determined in bespoke designs to satisfy subgrade rutting criteria are unlikely to be sufficient to meet the pavement deflection requirements that are detailed in Section 8.8.2 Austroads, 2017. Additional SIL and/or pavement material depths are likely to be required.

The ability to constructed subgrade improvement layers for existing roads requiring widening or pavement rehabilitation may be limited by the presence of underground utilities. If this is the case, then the catalogue designs are not appropriate, and alternative treatments should be detailed in a bespoke pavement design approach.

3.4.7 AASHTO Guides to Design Subgrade Resilient Modulus

Guidelines for 1993 AASHTO Pavement Design revised in 2003 provide following advice on deriving the resilient modulus for subgrade during pavement design.

Caution must be used when selecting a design resilient modulus of subgrade. An investigation and analysis of all the soils data should be conducted prior to selecting a designed subgrade CBR value. An average subgrade CBR should not be used as the design CBR if the variance of CBR tested is greater than 10%. In such case, the designer should look at sections with similar CBR values with less than 10% variance and design those section based on that average CBR tested or recommended by the geotechnical engineer.

If there are no sections clearly exist with similar test results, then it is advised to use a conservative subgrade CBR 10% of the values obtained for the design CBR.

For fine-grained soils with a soaked CBR between 5 and 10, use the following equation to correlate CBR to resilient modulus (Mr):

Design Mr = 10 x CBR

For coarse-grained soils with a soaked CBR greater than 10, use the following equation:

Design Mr = 15 x CBR $^{0.65}$

Typical values for fine-grained soils are 15 to 70MPa. Typical values for coarse-grained soils are70 to 138MPa. When FWD testing is conducted and the back calculated resilient modulus is determined, use the following equation:

Design subgrade modulus = 0.33 x Back calculated Resilient Modulus for Subgrade (Mr)

If the Design Mr based on CBR is greater than 10Mpai or if the Design Mr from back calculation is greater than 100MPa, then design subgrade modulus (Mr) value should not be more than 100MPa in any case or subgrade CBR 10%.

3.4.8 Pavement Drainage

Drainage of the pavement provided in Section 2.6 and Section 12.4 Subsoil Drains.

3.5 ENVIRONMENTAL EFFECTS IN MECHANISTIC / EMPIRICAL DESIGN

It has been acknowledged from several research findings that environmental conditions have a significant effect on the performance of both flexible and rigid pavements. External factors such as precipitation, temperature, and depth of water tables play a key role in defining the extent of impact the environment can have on the pavement performance. Internal factors such as the susceptibility of the pavement materials to moisture, infiltration potential of the pavement layers and performance of drainage will define the extent to which the pavement will react to the applied external environmental conditions.

Some of the effects of the environment on pavement materials are listed below:

- Asphalt bound materials (structural asphalt) including foamed bitumen materials exhibit varying modulus depending on the temperature. Modulus can go up to 10,000MPa or more in colder months to about 600MPa or less during summer months. Adaptation measures are provided in Section 1.5.
- All other conditions being equal, the higher moisture content the lower the modulus of pavement materials - unbound and cementitious materials due to high porewater pressure.
- Modulus of subgrade, especially cohesive soil are likely to be affected by clay-water-electrolyte interaction, which are complex phenomenon. Such mixture can also affect the structure of soil through reduction of cementation between the soil particles.
- Bound materials are not directly affected by presence of moisture, however excessive moisture can lead to stripping in asphalt mixtures causing a long-term effect on the structural integrity of cement bound materials.

All the above distresses affected by environmental factors such as seasonal fluctuation in the moisture and temperature profiles in the pavement structure brought by changes in the ground water table, precipitation / infiltration and other external factors are to be considered while carrying out the pavement design. Please note currently adopted mechanistic-empirical design procedures for pavement design need such factors to be considered especially while making design assumptions of the subgrade strength moduli or CBR.

3.6 PAVEMENT IMPACT ASSESSMENT

Heavy vehicle traffic movements generated by new developments either short-term or long-term can have a significant impact on the existing road especially on the pavements both during the construction of the development and operational phase when land development and building construction has been completed. This is especially required for all developments associated with quarry and land-fill development where large volume of heavy vehicle traffic movements are expected to cause significant stresses on to the pavement structure and therefore resulting reduced performance and its remaining service life.

Guide to Traffic Impact Assessment Practice Note: Pavement Impact Assessment. Brisbane: Queensland Government (Clarke, A 2018) and Department of Transport and Main Roads (2019). Guide to Traffic Impact Assessment (Department of Transport and Main Roads).

3.6.1 Asset Management Requirements

Auckland Transport has a duty in ensuring a safe and efficient network for all users. Where the road pavement is operating beyond its capacity, the condition will rapidly decline and may result in damage to the existing pavement and thus unsafe driving behaviours, or a lower operating speed may occur. Unplanned heavy maintenance works and renewals can also result in unnecessary road user disruptions. As such, Pavement Impact Assessment (PIA) is required to document the likely to impact of the development project with respect to:

- Impact on the current pavement design life (remaining pavement life calculation on the existing and projected new traffic loading)
- Any temporary strengthening requirements for the road section affected by the proposed development.

When a change in loading occurs, the remaining existing pavement life needs to be assessed and quantified to enable safe operation including decisions on mitigation measures and intervention treatment. The need for a PIA is triggered either when the expected development related HCV exceeds the current proportion of HCV in the zone of influence by 10% or when the expected development related vehicles result in a substantial change depending on the existing pavement strength and current level of use.

AT Asset Management considers that a change in traffic loading is considered a trigger for Pavement Impact Assessment (PIA) either when the consent related total HCV volume on both directions exceeds 10% of the current volume, or if the increase in consent related vehicles results in a substantial or permanent change in the road carriageway's One Network Road Classification (ONRC) requiring road upgrading including carriageway widening or introduction of new intersections / roundabouts.

PIA will be used to understand the impact due to proposed development activity and to inform Auckland Transport Asset Manager on road safety, maintenance and renewal programme and budgets. Main objective of the PIA is to compare and check the structural capacity and design life of the existing pavement including identifying intervention treatment for a good operation arising from the proposed additional traffic loading.

3.6.2 PIA Reporting

PIA report requires pavement modelling with back analysis to check the pavement conditions, remaining life and recommendation on pavement renewal or rehabilitation to be undertaken by a suitably qualified pavement engineer following the field assessment. The results of the PIA are tabulations of rehabilitation and maintenance over the analysis period due to the proposed development. Generally, the following steps outline the process for assessing the pavement impacts of a development:

- Current traffic including annual average daily traffic (AADT), percentage of commercial vehicles, growth rate and distribution of vehicles by class (if known) and likely number of ESAs per commercial vehicle.
- List and the number and types of vehicles that will be generated by the proposed development.
- Calculate the annual ESAs with and without the development based on information obtained from AT and NZTA WiM Data based in Drury Station.
- Current pavement age and current pavement design life.
- Predict when the pavement will require rehabilitation with and without the proposed development based on its remaining life and the forecast traffic.
- Cost of routine maintenance with and without proposed development
- Assess the pavement rehabilitation required at the end of the remaining life of the pavement with the current traffic, and with the current traffic plus the additional traffic generated by the development.
- Design details of any proposed rehabilitation.
- Establish if there is a change in the vehicle mix using the road that may require widening of the pavement or surfacing; and

- Predict the total cost of routine and programmed maintenance in each year to the design horizon with the current traffic, and with the current traffic plus the additional traffic generated by the proposed development.

As part of the Engineering Plan (EA) Application submission an assessment of the impacts of the increased heavy vehicle including construction traffic on existing road networks that the development connects to shall be made for determining developers' responsibility and contribution toward the maintenance.

A Pavement Impact Assessment Practice Note including the scope of PIA is still being developed by AT and Auckland Council for further reference and will be made available on the Council web site.

4.0 DESIGN TRAFFIC

4.1 DESIGN TRAFFIC INPUTS

Traffic counts on existing roads can be determined from Auckland Transport RAMM traffic data or traffic estimated / count from Mobile Road and Auckland Transports publicly available traffic count data spreadsheet. Similarly, Traffic modelling is generally undertaken as part of road upgrade works for existing or new roads as well as traffic counts undertaken as part of the project concept stage work with adjustments to traffic analysis using standard practice can be considered as a basis for determining the traffic numbers.

For new roads, projected traffic generated by the development needs to be determined, including percentage of likely heavy commercial vehicles and expected growth. This may require traffic modelling with M2 - secondary and district collector and M3 - regional / primary collector roads, to account for demand growth and the influence of adjacent developments.

4.1.1 Trip Generation

A standard and straightforward approach for estimating future trip generation volumes is to translate trends from the past into the future based on a linear growth trend of effective factors such as population or income. This method projects past data into the future by assuming a constant growth rate between two historical points.

The peak hour trip generation of dwelling houses is typically estimated using the predictive models within the RTA Guide1 or the NZTA Research Report 453 Trip and Parking Related Land Use. As the number of dwellings on-site and the scale of the retirement village is uncertain at this stage, the trips generated by the development cannot be calculated. As such, the intersections have been assessed to ascertain at which trip thresholds result in unsatisfactory operational performance. Further details can be referred to Austroads Guide to Traffic Management Part 12: Integrated Transport Assessments for Developments 2020.

In the absence of traffic modelling data to determine the generated traffic with estimated heavy vehicle movements, the estimation of traffic generated by a particular new development can be calculated for M1 - local roads using the New South Wales Roads and Traffic Authority (RTA) Guide to Traffic Generating Developments. It is recommended to use typical indicative residential daily vehicle trips (each direction of travel) from residential developments in this document in the absence of traffic modelling data:

- Low Density, separate dwelling houses – 11 trips per dwelling
- Medium Density, smaller units and flats (up to 2 bedrooms) – 7 trips per dwelling
- Medium Density, larger units and town houses (3 or more bedrooms) – 8 trips per dwelling

Care needs to be taken in selecting daily traffic and percent heavy vehicles when there will potentially be new adjacent developments connected to the road being designed. Future stages in developments need to be accounted for in the assessment of traffic for the current development being designed.

4.1.2 TLD Calculation

The Drury (decreasing direction) WiM data shall be used to model pavement truck loading in Auckland unless an alternative traffic loading configuration is appropriate for the design. The Drury Traffic Load Distribution (TLD) WiM data is available in Austroads AGPT-2 and is incorporated into CIRCLY 7 software. A csv file needs to be created to import it into AUSTPADS software.

If the TLD from Drury is not appropriate, then an alternative TLD can be created and used, provided it can be demonstrated as being suitable or more conservative. Alternatively, using the equivalent single axle (ESA) approach may be appropriate to account for concentrated loading or specific load configurations. How the load configuration has been developed will need to be demonstrated, nevertheless, for very low volume residential street with cul-de-sac or no exit and length less than 500m, the average number of HVAG per heavy vehicle to be used in the traffic calculation should not be less than 2.1 and the ESA/HVAG to be not less than 0.5, including low traffic volume roads with AADT <400 and HCV < 3%.

4.1.3 HCV Assumption

Where there is no classified count data available, HCV volume to be used is a minimum 3% for residential streets and a minimum 15% for industrial and commercial streets respectively of the estimated annual average daily traffic (AADT). For primary and regional arterial roads, a proper traffic analysis and modelling to be carried out to determine the forecast heavy vehicle traffic movements for the design period (minimum 25 years).

4.1.4 Bus Loading

In general, diesel and electric bus loadings are to be considered for the pavement design such as bus terminal including parking bays, and pavement with manor or rapid transit bus routes. A detailed movement analysis considering the bus type, occupancy or capacity, frequencies, and type of route to be carried out for major bus route to come up with equivalent single axle loading in accordance with the bus data such as tare weight and loaded axles to be provided by Auckland Transport or any other bus operators. For reference bus types and axle groups and their tare weight and maximum weight of buses currently in operation in the AT network are provided in the Appendix B. Bus types, tare and gross mass and routes travelled data can also be sourced from bus companies operating on (or expected to operate on) the pavements to be designed. In the absence of local project data, the Auckland Transport database may be used.

4.1.4.1 Bus Types and Axle Group

Buses typically come with three axle group arrangements in the Auckland regions:

- Smaller single deck buses have a single axle single tyre (SAST) front steering axle and a single axle dual tyre (SADT) rear axle.
- Larger single deck buses have a SAST front steering axle and an unusual rear axle group consisting of a SADT in front of a SAST tag axle.
- Double deck buses are like single decker with a SAST front steering axle and an unusual rear axle group consisting of a SADT in front of a SAST tag axle.

There are no articulated buses used in Auckland.

The rear axle group of the buses that are currently in use in Auckland consists of the SADT - SAST tag axle group and therefore is not a standard axle group in the Austroads AGPT Part 2 Chapter 7, although these buses are compliant to Land Transport Rules - Vehicle Dimension and Mass Rule (VDMA) published by Ministry of Transport. Therefore, the two axles in this group should be treated separately when creating a traffic load distribution for pavement design. The typical distribution of load on the SADT – SAST tag group is 55% on the SADT, 45% on the SAST tag.

In addition, the equivalent axle load to be calculated based on the number of repetitions of a standard axle that are equivalent in damaging effect on a pavement for a given axle group type and loading calculated with a load damage exponent of 4 (Austroads quadrant equation) for all pavement types including concrete pavement.

Generally, AT Public Transport programme can be used to determine the bus route, bus type and frequencies including peak and off-peak volume of buses. However, in absence of such data other than projected volume of bus movements and for simplification for single deck diesel and or electric bus operation, it is recommended to use 50% split in between 3-axle and 2-axle buses and 50% occupancy for TLD calculation on general bus route. TLD to be calculated for a design life of 25 years with up to 2% growth.

When designing a specific bus operational pavement, a bespoke traffic load distribution is required. Normally, it is suggested to provide a bus TLD to be used in Appendix B for flexible pavement design and rigid pavement design. However, maximum bus loading should be considered for concrete pavement design.

4.1.4.2 Bus Weight / Loading Data

With high emphasis on public transport due to congestions issues in urban town centres, the bus fleet and proportion of double-decker electric buses is likely to continue to grow. This will increase the average axle loadings on pavements.

As such, pavement design especially for new projects especially bus terminal and bus parking bay and major bus rapid transit routes to be carried out TLD calculation by analysing the current and future bus frequencies, type of bus – 2 axle, 3 axle or double decker – 3 axle buses.

The following 3-types of buses are being identified to be considered in the pavement design:

1) 2-axles bus with single deck - front steering axle with single tyre and rear axle with dual tyres; capacity 32 seated and standing 8 passengers with total 40 passengers.
2) 3- axles bus with single deck - front steering axle with single tyre and front-rear axle with dual tyres and back-rear axle with single tyre (middle axle with dual tyres); capacity 35 seated and standing 20 with total maximum 55 passengers.
3) Double decker 2021: 3 -axles front steering axle with single tyre and rear tandem axle with single tyres; Capacity 35 plus 7 standing (Lower Level) 55 seated (Upper Level) - (Total 97 passengers)

Mass loadings on bus axles vary and a details configuration including tracking information for typical e-bus used in the AT network is provided in the Appendix B.

4.1.4.3 Bus TLD for Pavement Design

For flexible pavements for routes where buses form a part of the general traffic a standard TLD may be sufficient, or if the bus numbers and axle loads can be determined, two traffic load distributions could be applied for routes where the TLD is unknown, a bus TLD and a general traffic TLD. In the latter case the two TLDs can be modelled separately, and the results are combined to check damage factors. For ESA designs an assessment of the equivalent ESA for bus axles following the procedures in AGPT Part 2 Chapter 7 shall be carried out.

Since the magnitude of bus loading is critical, it is advised to construct bus stops and stations with concrete pavements. In selecting a traffic load distribution for a concrete pavement design, it is necessary to capture the proportion of heavier axle loads in the TLD or maximum weight of the bus on full load and a bespoke TLD shall be created for these pavements. It is recommended that for concrete pavement design the TLD created for 100% occupancy levels and maximum bus axle weights.

Appendix B provides currently available detailed bus design, capacity, axle type, tare and maximum weight for designer reference to model for pavement design. It is essential to find out the bus operation – frequency and type of buses used on the route including likely number of passengers during peak and off-peak hours. Generally, it is advised to take 50% full capacity in absence of the data, however a sensitivity check to be calculated if peak hour movements could govern the design.

4.2 TRAFFIC LOAD CALCULATION

Axles load from especially from heavy vehicles over a pavement generate vertical stresses and strain – resilient and permanent within the pavement that are transferred to the subgrade. Resilient strain deforms the pavement when the axle load is applied, but rebounds when the load is removed. However, with permanent strain, the deformation remains after the axle load has passed. While this could initially support compaction, over time the road starts to deform defined as rutting and typically, the incremental damage will quickly accelerate as traffic loading increases. The level of expected traffic movements for various heavy vehicle types must be estimated when designing a pavement.

A wide variety of axle spacing, and configurations of tyre and axle sets are used for heavy vehicles working within the transport industry. To undertake pavement design, these configurations need to be simplified into a small group of standard axle sets and loadings.

4.2.1 Loads on Axle Group

For design purposes, axle group loadings are generally simplified in terms of the vehicle types and their axle loads causing equivalent damage are as follows as provided in Austroads AGPT-2:

- Single axle with single tyre (SAST) – 53 kN
- Single axle with dual tyre (SADT) – 80 kN
- Tandem axles with single tyre (TAST) – 90 kN
- Tandem axles with dual tyre (TADT) – 135 kN
- Triaxles all with dual tyre (TRDT) – 181 kN
- Quad axle with dual tyre (QADT) - 221kN

The conversion to standard axles uses an exponent of four. The consequence is that axle groups with actual loads greater than the standard load will cause significantly greater pavement damage. The above loadings are based on 375mm to 450mm tyre section width; for larger single tyre section width 450mm or more, it is advised to refer Austroads AGPT-2 (2027) Table 7.8.

4.2.2 Construction Traffic

Construction traffic for new developments and their development buildings shall be calculated separately to the future road operational traffic and added to it to calculate the total design traffic. This should not be a random percentage increase on the operational traffic period traffic increase.

In absence of detailed analysed data, construction traffic should be calculated based on an estimate of 2.5 ESA average per heavy vehicle and 10 heavy vehicles per dwelling with construction duration up to 2 years. For larger multi-story buildings, a more accurate estimate should be provided. Concentrated loading by construction traffic should also be accounted for in the traffic load calculation. Evidence for the values adopted in the design should be provided.

The design traffic must account for the additional HCV associated with current and known planned future construction projects and any high productivity motor vehicle (HPMV) trips such as specialised HCV vehicles and or double decker or newer electric buses, which have significantly higher axle loadings. Most of the buses, in use on the Auckland network, have axle configurations with tag axles that incorporate single axle single tyres and single axle dual tyres in a tandem axle configuration. Where appropriate the loading from these vehicles should be evaluated and incorporated into the design in addition to the standard axle loads. The type of the buses and axle loads to be used in the design is provided in the Appendix B and updated information can be made available from the AT web site.

4.2.3 Direction and Lane Distribution Factor (DF/LDF)

A direction factor (DF) of 0.5 is appropriate for more than two lanes with two direction of major road, unless there is a predominance of heavy vehicles in one direction, in such cases, it is advised to consider a higher number DF of 0.7 or more.

On narrow residential streets or minor roads with carriageway width less than 6.50m between the sealed edge or lip of concrete channel (or additional width parking bay edge), heavy commercial vehicles in each direction will tend to share a wheel path in the road centre area due to maintaining a shy distance from kerbs or parked vehicles along the road including parking bays. In this case the DF should be taken as 1.0.

A lane distribution factor (LDF) of 1.0 for each lane shall be used for both one and two lanes in each direction. Intersection design should consider the potential loading from all roads connected to the intersection including increased traffic loading due to turning, braking, stopping and acceleration movements.

4.2.4 Design Life

The design life for pavement design should be as follows:

- 25 years for flexible pavement including structural asphalt concrete pavement.
- 30 years for flexible pavement on major capital work projects especially for busway and associated pavement upgrade associated with rapid transit network development.
- 40 years for rigid (concrete) pavement; this includes bus bays, bus terminal and depot, concrete carriageway pavement, parking areas, raised safety platforms, roundabouts, and intersections with high turning traffic.

P3/M3 sites or the regional arterial and primary collector routes, and major rapid transit routes, as discussed in Section 1.7, will require the economics of the treatment calculated over an analysis period of 40 years unless there is a specific requirement by Council during the resource consent approval. Where required, the Monetised Benefits and Costs Manual[2] (2021) shall be used to evaluate the economics of the various possible pavement treatment options.

4.2.5 Growth Factor

Growth factor for use in design should be taken from traffic models or estimated based on the road connectivity to the wider network including developments that may connect to it in future. A default value of 3% should be considered as a minimum for new developments when a justified growth figure cannot be made. The exception to this is residential cul-de-sacs that are being fully developed in housing density, where a nominal growth factor of 1% is acceptable. For existing road network widening or upgrade, the adopted growth rate shall be estimated based on historic traffic data or calibrated traffic models but should not be less than 3%.

4.2.6 Traffic Load Calculation on Roundabouts and Intersections

Designers are required to consider additional shear and bearing forces applied to the pavement by vehicles as they traverse roundabouts and intersections. Forces from acceleration, braking and turning can be significant, hence NZTA's Technical Advice Note #17-01, Asphalt depths at high stress locations for new pavements and renewals[3], published in 2017. This requires structural asphalt pavements with minimum depth 125mm (depth to be determined by mechanistic modelling) as follows:

- For roundabouts with 100 or more heavy vehicles per lane per day, and
- For intersections (both signalised and non-signalised) with 500 or more heavy vehicles per lane per day.

Note for intersections, the calculation of heavy vehicles per lane per day shall include for areas where the wheel paths of multiple directions of vehicle travel cross. Such pavement should be extended to approaches to the intersections and roundabouts.

In addition, the effect of additional shear and bearing forces can be modelled within the mechanistic-empirical design process (Chapter 8 of Austroads 2017). The effect of heavy vehicle load transfer or additional intensity during turning is to be accommodated at the design traffic loading calculation stage by increasing the magnitudes of anticipated axle wheel loads by 30% (equivalent to an ESA or N_{DT} multiplier of 2.86 for roundabouts) and by 20% (equivalent to an ESA or N_{DT} multiplier of 2.07 for intersections and approaches to intersections and roundabouts). The approach lengths for the roundabout or intersection using structural asphalt concrete applies to is:

- For posted speed 50 kph and less, minimum 40m before the stop or give-way limit lines. and
- For greater than posted speed 50kph, 60m before the stop or give-way limit lines.

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[2] https://www.nzta.govt.nz/assets/resources/monetised-benefits-and-costs-manual/Monetised-benefits-and-costs-manual.pdf
[3] #17-01 Asphalt depths at high stress locations for new pavements and renewals | Waka Kotahi NZ Transport Agency (nzta.govt.nz)

- For departures or exit from intersections or roundabouts regardless of the posted speed, a minimum of 10m from the intersection/roundabout exit, or to the point where the outside kerb alignment becomes straight, whichever is the greater distance.

This requirement is considered for all places (P) and movements (M) classification of roads, when the heavy vehicle movement numbers as above: 100 HCV and 500 HCV or more per day per lane for roundabout and intersections respectively regardless of their road classification / hierarchy regardless of the design is for pavement rehabilitation or for a new construction.

4.3 TRAFFIC LOAD DESIGN CALCULATION

The design traffic for use in the pavement design should be determined using the below equations.

The parameters in the equation are defined in Austroads AGPT-2.

$$N_{DT} = 365.CGF.AADT.DF.\frac{\%HV}{100}.LDF.N_{HVAG}$$

$$DESA = 365.CGF.AADT.DF.\frac{\%HV}{100}.LDF.N_{HVAG}\cdot\frac{ESA}{HVAG}$$

Traffic load calculation in accordance with Austroads is presented for a range of traffic levels and the estimated traffic ranges are presented in Table 9 below.

The ESA/HVAG and N_{HVAG} used in the calculations are based on the Drury decreasing direction WiM data from Austroads AGPT-2. This is considered reasonable for the general traffic loading in Auckland, however, if a site has an unusual traffic configuration, for example a bus route or quarry traffic, then additional efforts to characterise the traffic will be required. For Drury, the ESA/HVAG is set at 0.605 while the N_{HVAG} is set at 2.61.

HPMV heavy vehicles, single and double decker electric buses with overweight permits, will need to be considered in the modelling and bescope pavement design to be carried. The designer will need to state the assumptions made for specialised or HPMV vehicles in the design report and a bespoke pavement design will be required.

These example calculations do not include any construction traffic, which would need to be included in addition to the normal post-construction road traffic. Intersection and roundabout pavement design should consider the additional traffic load from all roads connected to the roads as described above to account for the increased shear requirements for turning and braking on the higher volume intersections and roundabouts.

Table 9: Example Design Traffic Calculations

Factor	Value			
Assumed Design Period years	25-years			
AADT (total for both directions)	238	1188	3169	5942
HCV %	2	2	3	8
Annual Geometric Growth Rate %	3			
Direction Factor	1[1)]	1[1)]	0.50[2)]	0.50[2)]
Lane Distribution Factor	1			
Cumulative Growth Factor	36.5			
Axle Groups per HV, N_{HVAG}	2.61			
Design Traffic, N_{DT}, in HVAG	1.66×10^5	8.27×10^5	1.66×10^6	8.27×10^6
ESA per heavy vehicle axle group ESA/HVAG	0.605			
Design equivalent standard axles - DESA	1.00×10^5	5.00×10^5	1.00×10^6	5.00×10^6

Note: 1) carriageway width less than 6.5m between edges of sealed surface and or lip of the channels
2) single carriageway with flushed median width more than 6.5m between edges of sealed surface

5.0 CATALOGUE PAVEMENT DESIGN

This section outlines the various pavement types and recommends minimum design thicknesses for Catalogue Pavement Designs. These pavement designs do not allow for the electric bus axle loading, for which designer shall undertake design calculation and submitted under bespoke design.

5.1 GENERAL

A catalogue design is adopted for a range of traffic levels and subgrade conditions to generate several of the pavement designs suitable for a catalogue design approach. The M1 and M2 movement categories are expected to have an average daily traffic, in both directions, up to 10,000 vehicles per day.

Designs for granular pavements are given in Section 5.1.1, cement bound subbase pavements in Section 5.1.2, foamed bitumen pavements in Section 5.1.3, structural asphalt pavements in Section 5.1.4 and concrete pavements in Section 5.3.5.

Subgrade strengths in catalogue designs are based on the soaked CBR strengths of natural subgrade found in almost all situations in the Auckland region as follows:

- Soaked CBR 2% to 3% with a contingency plan for CBR < 2%
- Soaked CBR 3.5% or greater with a contingency plan for less than CBR 3.5%
- Soaked CBR 5% or greater (usually engineered filled with depth > 1,5m) with a contingency plan for less than CBR 5%

Distresses affected by environmental factors such as seasonal fluctuation in the moisture and temperature profiles in the pavement structure brought by changes in the ground water table, precipitation / infiltration and other external factors are to be considered while carrying out the pavement design especially while making design assumptions of the subgrade strength moduli or presumptive CBR values.

These catalogue designs have been calculated for P1/M1 (Accessway / Local Roads), P1/M2 and P2/M1 (Residential Streets) without any bus route traffic classification with a design reliability of 90%. Based on Table 5: Design Reliability required for Various Combinations of Movement and Place Significance if the road has a P2 or P3 in combination of M2 and M3 categorisation then a design reliability of 95% is required.

A bespoke design is required for weak subgrade with CBR < 2% and higher design traffic loading on ESA and primary collector and arterial roads with major bus routes and any concrete pavement design.

For all new pavements in urban setting, the finished surface is to be thin asphalt over a membrane seal at residual binder 1.4 litre/m^2. If the road will have traffic before final surfacing with asphalt, other than limited light construction vehicles, then the membrane seal should be replaced with a two-coat chip sealing using grade 3 and 5 chips. A tack coat shall be placed immediately prior to placing the thin asphalt wearing course. It is essential to allow additional depth of two-coat chip sealing into consideration while the pavement construction is commenced from subgrade level. The thin asphalt wearing course shall be compliant with NZTA M10. It is recommended to use AC14 or AC10 of PG64 V or PG64 E for primary collector and arterials and DG10 or AC10 PG64 S grade performing binder for local roads with traffic less than 1.0×10^6 ESA.

Refer to Table 24 for detailed information on asphalt concrete surfacing with recommended performing grade binder.

5.1.1 Granular Pavements

Granular material is defined as material meeting the requirements of AT Aggregate Specifications or NZTA M04 basecourse aggregates. Granular material proposed as an upper unbound or cement treated basecourse for overlaying with asphalt concrete including surfacing shall consist of AT AP40 or basecourse as per NZTA M4 AP40 aggregates or better.

All base and subbase course comprising crushed rock shall be sublayered in accordance with the Austroads AGPT-2 sub-layering procedure. The design vertical modulus value in the vertical direction at the top of the layer lower subbase layer shall not exceed 150 MPa and top basecourse shall not exceed 350 MPa.

A Poisson's Ratio value of 0.35 shall be used for all granular material. Granular materials shall be considered as anisotropic with a degree of anisotropy of 2.

Subject to approval by AT, some naturally occurring gravel pavement materials may be considered as a granular material for use as lower subbase in lieu of crushed rock. However, the maximum design vertical resilient modulus value at the top of a gravel layer shall be ≤ 150 MPa.

For design traffic values up to 1×10^6 ESA, granular pavement option for catalogue designs are given in Table 10, Table 11 and Table 12. These designs are based on Figure 8.4 designs from Austroads AGPT-2 and compliance with Benkelman Beam deflection and deflection curvature (D_0-D_{200}) criteria shown in Table 25 and associated traffic levels in Table 9. Note, the Austroads AGPT-2 Figure 8.4 shows minimum basecourse thickness 100mm, whereas this TDM recommend increasing to 125mm minimum to improve constructability especially with compaction while reducing the subbase thickness by 25mm, making overall depth the same.

Deflection compliance uses the following road types for design traffic. Refer Section 6, deflections.

- Up to 100,000 ESA, Local Road (Residential Streets).
- 100,000 to 500,000 ESA, Collector / Rural Arterial without buses.
- 500,000 to 5,000,000 ESA, Collector / Arterial with buses

For lower subgrade strength CBR 2 to 3%, it is recommended for subgrade stabilisation either using lime and or cement additives for which quantity is to be confirmed following a laboratory reactivity testing. An alternative option is to place black sand (Woodhill sand) fully encased in a TNZ F7 - Class C geotextile.

For example, for a Design CBR of 2 to 3% with Design Equivalent Standard Axles up to 1×10^5 ESA the total pavement thickness shall be a minimum 695mm including subgrade improvement, whereas, for a Design CBR of 2% and a DESA from 5×10^5 to 1×10^6 the total pavement thickness shall be 800mm. If subgrade stabilisation with chemicals is adopted, soaked CBR testing in an accredited laboratory is required to determine chemical dosage to achieve a minimum laboratory soaked CBR of 15%. It is advised to refer to NZTA M15 Specification for Lime for use in Soil Stabilisation on supply of calcium oxide calcium hydroxide for use in soil stabilisation and sampling and testing of additives for quality assurance.

Refer ASTM D6276 Using pH to estimate the Soil-Lime Proportion Requirement for Soil Stabilisation - Lime Demand Test, to confirm the minimum percentage lime. This test should include any cement contents proposed.

Table 10: Catalogue Granular Pavement Designs for Subgrade with a CBR 2 to 3%

Layer Description	Design traffic (DESA)		
	Up to 1×10^5	From 1×10^5 to 5×10^5	From 5×10^5 to 1×10^6
Minimum AT AP40 or TNZ M4 AP40 Basecourse (mm)	125 [1]	150	200
Minimum AT AP65 Subbase Course (mm)	240	320	400
Minimum Subgrade Improvement Layer (mm) Stabilised Subgrade or Black Sand [2]	300	300	300
Total minimum thickness (mm)	665	770	800

1. Minimum depth of basecourse to be 125mm
2. Subgrade improvement layer can be:
   i. Chemically stabilised in-situ or imported subgrade using lime and/or cement (laboratory soaked CBR testing required to determine dosage to achieve minimum laboratory soaked CBR of 15%)

ii. Mechanically stabilised by importing black sand, encased in Class C geotextile.

In the event a subgrade of CBR <2% is found on site, natural material or resulting from construction activities, an additional subgrade improvement layer of 500mm is required. Use the CBR 2% designs in Table 10 with a black sand subgrade improvement layer and increase the thickness of the subgrade improvement layer by 500mm of black sand wrapped in Class C geotextile. For CBR 2% designs this CBR 1% contingency plan for weak areas shall be shown on the Drawings.

Table 11: Catalogue Granular Pavement Designs for Subgrade with CBR = 3.5%

Layer Description	Design traffic (DESA)		
	Up to 1×10^5	From 1×10^5 to 5×10^5	From 5×10^5 to 1×10^6
Minimum AT AP40 or TNZ M4 AP40 Basecourse (mm)	125	150	200
Minimum AT AP65 Course (mm) - 2 layers	340	400	460
Subgrade Improvement Layer [1]	Class C Geotextile & Triaxial Geogrid [2]	Class C Geotextile & Triaxial Geogrid [2]	Class C Geotextile & Triaxial Geogrid [2]
Total minimum thickness (mm)	465	550	660

1. Class C geotextile separation layer to stop fines migration from subgrade for strength less than CBR 3.5%
2. Triaxial Grid specify what type or reference standard?

In the event a subgrade of < CBR 3.5% is found on site, natural material or resulting from construction activities, a contingency plan using the CBR 2 to 3% designs in Table 10 shall be shown on the Drawings.

Table 12: Catalogue Granular Pavement Designs for Subgrade Improvement achieving a CBR ≥ 5%

Layer Description	Design traffic (DESA)		
	Up to 1×10^5	From 1×10^5 to 5×10^5	From 5×10^5 to 1×10^6
Minimum AT AP40 or TNZ M4 AP40 Basecourse thickness (mm)	125	150	200
Minimum AT AP65 Subbase Course thickness (mm)	275	310	360
Total minimum thickness (mm)	400	460	560

In the event a subgrade of < CBR 5% is found on site, natural material or resulting from construction activities, a contingency plan using the CBR 3.5% designs in Table 11 shall be shown on the Drawings.

5.1.2 Cement Bound Subbase Course Pavements

Pavements with a cement bound subbase have lower deflections and, therefore, economy in pavement thickness can be achieved. To ensure the design strength of the subbase is achieved, construction traffic and early loading from development traffic will need to be considered in the design and potentially kept from the subbase until sufficient strength has been developed.

Cement stabilisation should not be carried out for in-situ subgrade CBR < 3% unless the subgrade has been improved with chemical stabilisation and or mechanical stabilisation with black sand. For subgrade strength CBR 3.5% a Class C geotextile on the subgrade is required under the cement stabilisation to prevent fines migration.

Cement stabilisation of subbase course can be by using either plant mix or in-situ stabilisation with an unbound granular buffer thickness 50mm minimum to prevent hoeing of the subgrade or geotextile.

As detailed in Section 2.6 Subsoil Drains, the minimum thickness of unbound basecourse under the kerb and channel shall be 50 mm, and it must drain to the pavement drainage system to enable discharging sub-surface water (either open side drain or subsoil drain), therefore, the basecourse thicknesses detailed below are based on allowing this drainage path. Where subsoil drains are included the subbase course stabilisation must terminate at the subsoil trench. This allows any infiltration through the cracks between channel and pavement surface and any trapped moisture within the basecourse layer to reach and reticulate sub-surface water into the subsoil drain, which connects to unbound aggregate of the basecourse layer.

The thickness of dense graded asphalt (including SMA) and/or unbound granular material to be placed over cementitious treated material (CTB) shall be ≥ 175 mm and determined in accordance with Austroads as follows:

Thickness = (0.75 x unbound granular thickness + dense graded asphalt including surfacing thickness)

An unbound basecourse is recommended above the cement bound subbase course, provided the moisture levels can be controlled, and this will likely mean not constructing during the wet months. Where moisture levels are likely to impact the workability and performance of the unbound aggregate, light cement modification might be considered to reduce moisture susceptibility. If required, the cement content when modifying the basecourse must not exceed 1.5% or generate a dry ITS greater than 350 kPa when tested in accordance with NZTA T19.

Cement stabilisation of the subbase will likely require cement content between 3 and 4%, the design parameters discussed in Section 11.5 and Section 11.6 should be considered in determining the necessary cement content. If premixed cement is used, compaction of the pavement must be completed within 2 hours of the mixing of the cement and water. Trafficking of the subbase shall be minimised to allow only traffic necessary to construct the subbase course and the overlying basecourse. The cement bound subbase has been modelled as a fatigue cracked cemented aggregate.

Table 13: Cement Bound Subbase for Subgrade with CBR 2 to 3%

Design traffic (DESA)	From 1×10^5 to 5×10^5	From 5×10^5 to 1×10^6	From 1×10^6 to 5×10^6
For finished with chip sealed surface: Minimum AT AP40 or TNZ M4 AP40 basecourse (mm) [1]	230 [1]	230 [1]	230 [1]
For thin asphalt concrete surface – 40mm DG10 / AC10 Minimum AT AP40 or TNZ M4 AP40 basecourse (mm) [1]	190 [1]	190 [1]	190 [1]
Minimum AT AP40 or TNZ M4 AP40, 3-5% cement bound [2]	200	230	250
Minimum Subgrade Improvement Layer (mm) Stabilised Subgrade or Black Sand [3]	300	300	300
Total minimum thickness (mm) for Chip sealed	730	760	780
Total minimum thickness (mm) excluding thin asphalt surface - 40mm DG10 / AC10	690	720	740

1. Depth of basecourse is based on 180mm kerb and channel depth plus 50mm for drainage underneath for chip sealed or asphalt surface finished pavement.
2. Thickness shown is depth of material for in-situ stabilisation. Depth of stabilisation to be 50mm less than depth shown. For premixed cemented subbase, depth of layer shown can be reduced by 50mm minimum.
3. Subgrade improvement layer can be:
   i. Chemically stabilised in-situ or imported subgrade using lime and/or cement (laboratory soaked CBR testing required to determine dosage to achieve minimum laboratory soaked CBR of 15%)
   ii. Mechanically stabilised by importing black sand, encased in Class C geotextile.

In the event a subgrade of CBR 1% is found on site, natural material or resulting from construction activities, an additional subgrade improvement layer of 525mm and additional cement bound layer of 150mm is required. Use the CBR 2% designs in Table 13 with a black sand subgrade improvement layer and increase the thickness of the cemented layer to 300mm and the subgrade improvement layer to 800mm of black sand wrapped in Class C geotextile. For CBR 2% designs this CBR 1% contingency plan for weak areas shall be shown on the Drawings.

Table 14: Cement Bound Subbase for Subgrade with CBR 3.5%

Design traffic (DESA)	From 1×10^5 to 5×10^5	From 5×10^5 to 1×10^6	From 1×10^6 to 5×10^6
For finished with Chip sealed surface: Minimum AT AP40 or TNZ M4 AP40 basecourse (mm) [1]	230 [1]	230 [1]	230 [1]
For thin asphalt concrete surface – 40mm DG10 / AC10 Minimum AT AP40 or TNZ M4 AP40 basecourse (mm) [1]	190 [1]	190 [1]	190 [1]
Minimum AT AP40 or TNZ M4 – AP40 subbase course, 3-5% cement bound [2]	200 [2]	240 [2]	300 [2]
Subgrade improvement layer [3]	Class C Geotextile		
Total minimum thickness (mm) – Chip sealed	430	470	530
Total minimum thickness (mm) – Thin asphalt	390	430	490

1. Depth of basecourse is based on 180mm kerb and channel depth plus 50mm for drainage underneath.
2. Lower thickness by 50mm if premixed cement bound, higher thickness if hoeing cement in-situ (25mm buffer layer of unbound granular to mitigate risk of hoeing of subgrade) for in-situ stabilisation).
3. Class C geotextile separation layer to stop fines migration from subgrade for strength less than CBR 4%

In the event a subgrade of < CBR 3.5% is found on site, natural material or resulting from construction activities, a contingency plan using the CBR 2 to 3% designs in Table 13 shall be shown on the Drawings.

Table 15: Cement Bound Subbase for Subgrade with CBR ≥ 5%

Design traffic (DESA)	From 1×10^5 to 5×10^5	From 5×10^5 to 1×10^6	From 1×10^6 to 5×10^6
For finished with Chip sealed surface: Minimum AT AP40 or TNZ M4 AP40 basecourse (mm) [1]	230 [1]	230 [1]	230 [1]
For thin asphalt concrete surface – 40mm DG10 / AC10 Minimum AT AP40 or TNZ M4 AP40 basecourse (mm) [1]	190 [1]	190 [1]	190 [1]
Minimum AT AP40 or AT AP40CC subbase, 4% cement bound [2]	150	150	170
Total minimum thickness (mm) – Chip sealed	380	380	400
Total minimum thickness (mm) – Thin asphalt surface	320	340	360

1. Depth of basecourse is based on 180mm kerb and channel depth plus 50mm for drainage underneath.
2. 'Thickness shown is depth of material for in-situ stabilisation. Depth of stabilisation to be 50mm less than depth shown. For premixed cemented subbase, depth of layer shown can be reduced by 50mm minimum.

In the event a subgrade of < CBR 5% is found on site, natural material or resulting from construction activities, a contingency plan using the CBR 3.5% designs in Table 14 shall be shown on the Drawings.

5.1.3 Foamed Bitumen Stabilised Pavements

Foamed bitumen stabilisation of the basecourse is mostly appropriate for rehabilitation or strengthening of existing pavement and it provides a good platform for thin asphalt surfacing to perform. Generally, a foamed bitumen stabilised basecourse results in a low curvature pavement and being a stiffer pavement, it has relatively low deflections.

The following catalogue design treatments are for Greenfield pavements, pavement rehabilitations using foamed bitumen and imported, or the existing tested aggregates will require a bespoke pavement design. The foamed bitumen stabilisation mix design should be performed on the proposed aggregate prior to construction following the methodology detailed in the New Zealand Guide to Pavement Evaluation and Treatment Design or latest NZTA Waka Kotahi Pavement Design Guide to Austroads Supplement, which is being expected to release in 2026.

Foamed bitumen stabilised pavement catalogue designs are given in Table 16, Table 17, and Table 18 for subgrade CBR 2 to 3%, 3.5% and 5% respectively. For much weaker subgrade with CBR < 2%, it is recommended to submit a bespoke design with a detailed geotechnical investigation report.

Table 16: Foamed bitumen Catalogue Pavement Designs for Subgrade with a CBR of 2% to 3%

Design traffic (DESA)	From 1×10^5 to 5×10^5	From 5×10^5 to 1×10^6	From 1×10^6 to 5×10^6
Minimum foamed bitumen stabilised basecourse AT AP40 or TNZ M4 AP40 (mm)	170	180	230
Minimum unbound subbase GAP65 (mm). ATAP40 or M4 AP40	170	170	200
Minimum Subgrade Improvement Layer (mm) Stabilised Subgrade or Black Sand [3]	300	300	300
Total minimum thickness (mm)	640	650	730

In the event a subgrade of CBR 1% is found on site, natural material or resulting from construction activities, an additional subgrade improvement layer of 525mm and additional unbound subbase layer of 150mm is required. Use the CBR 2% designs in Table 10 with a black sand subgrade improvement layer and increase the thickness of the unbound subbase course by 150mm for the traffic level being designed, and the subgrade improvement layer to 800mm of black sand wrapped in Class C geotextile. For CBR 2% designs this CBR 1% contingency plan for weak areas shall be shown on the Drawings. However, we recommend preparing a bespoke design for subgrade CBR < 2%.

Table 17: Foamed Bitumen Catalogue Pavement Designs for Subgrade with CBR 3.5%

Design traffic (DESA)	From 1×10^5 to 5×10^5	From 5×10^5 to 1×10^6	From 1×10^6 to 5×10^6
Minimum foamed bitumen stabilised basecourse (mm)	170	180	230
Minimum unbound subbase (mm)	300	300	300
Subgrade improvement layer [1]	Class C Geotextile		
Total minimum thickness (mm)	470	480	530

1. Class C geotextile separation layer to stop fines migration from subgrade for strength less than CBR 4%

In the event a subgrade of < CBR 3.5% is found on site, natural material or resulting from construction activities, a contingency plan using the CBR 2 to 3% subgrade improvement layer in Table 17, in addition to the thicknesses in Table 17 shall be shown on the Drawings.

Table 18: Foamed Bitumen Catalogue Pavement Designs for Subgrade with CBR ≥ 5%

Design traffic (DESA)	From 1×10^5 to 5×10^5	From 5×10^5 to 1×10^6	From 1×10^6 to 5×10^6
Minimum foamed bitumen stabilised basecourse (mm)	150	170	200
Minimum unbound subbase (mm)	200	200	200
Total minimum thickness (mm)	350	370	400

In the event a subgrade of < CBR 5% is found on site, natural material or resulting from construction activities, a contingency plan using the CBR 3.5% subgrade improvement layer in Table 17, in addition to the thicknesses in Table 18 shall be shown on the Drawings.

5.1.4 Structural Asphalt

The design of deep-lift and full depth asphalt pavements shall be based on the mechanistic – empirical design procedures in accordance with Austroads AGPT-2 and as per this TDM. The pavement response to load shall be calculated using a linear elastic model, such as that provided by the computer programs CIRCLY and AustPADS.

The pavement response to loading as shown in Figure 8.2 of Austroads AGPT-2 shall be determined. Critical locations in the pavement for the calculation of strains resulting from an axle with:

- dual tyres, shall be on vertical axes through the centre of an inner tyre load and through the point midway between the two tyre loads.
- single tyres, on a vertical axis through the centre of the tyre.

5.1.4.1 Weighted Mean Annual Pavement Temperature

Considering the increase in temperature due to climate change, the Weighted Mean Annual Pavement Temperature (WMAPT) for all new roads in the Auckland region should be taken as 25°C. This means, there is no need for temperature adjustment to the laboratory tested resilient modulus values. Similarly, for pavement rehabilitation and strengthening and resurfacing of the existing carriageway, the Weighted Mean Annual Pavement Temperature (WMAPT) shall be taken as 25°C unless otherwise specified by Auckland Transport.

5.1.4.2 Design Speed

The design speeds adopted for catalogue design are:

- 10 kph to allow for controlled intersections, roundabouts, raised safety platforms.
- 10 kph to allow for mid-block pavement design with 30 kph posted speed zone.
- 40 kph to allow for mid-block pavement design in 50kph and 60kph zones
- 70kph to allow for mid-block pavement design in greater posted speed up ranging from 70kph to 80kph where asphalt macro-texture to comply with NZTA T10.
- Bus terminals and bus parking area and therefore use of a conservative presumptive design modulus at near zero speed to be taken into consideration of static and stationery loading.

5.1.4.3 Design Parameters

The asphalt types used, and Shell Fatigue Equation model parameters used for catalogue and bespoke designs are provided in the Table 19. These are the values from the AT approved asphalt mixes from a range of suppliers calculated based on recent laboratory results carried out for the mix design. The asphalt design parameters – presumptive resilient modulus 'E' and Shell coefficient 'k' have been adjusted for a design traffic at 10kph and 40kph with WMAPT of 24-degree Celsius. Constructed air voids for asphalt of 5% has been adopted for structural pavement design for both catalogue and bespoke design.

In situations where the cemented layer is a subbase beneath a granular or asphalt thickness greater than or equal to 175mm, the post-fatigue cracking life may be estimated as detailed in Austroads AGPT-2 Section 8.2.6.

Table 19: Asphalt Model Parameters for Catalogue and Bespoke Pavement Design

Asphalt Type	Design Speed 3kph (Bus Terminal)	Design Speed 10 kph	Design Speed 40 kph	Design Speed 70 kph
DG10 wearing course	Not used.	k = 0.005767 E = 1360 MPa	k = 0.004806 E = 22650 MPa	Not used.
AC10 wearing course	k = 0.007196 E = 780 MPa	k = 0.006143 E = 1210 MPa	k = 0.00512 E = 2010 MPa	k = 0.004757 E = 2460 MPa
AC14 wearing course	k = 0.006450 E = 870 MPa	k = 0.005506 E = 1360 MPa	k = 0.004589 E = 2250 MPa	k = 0.004264 E = 2760 MPa
AC20 base course	k = 0.005320 E = 1100 MPa	k = 0.004542 E = 1710 MPa	k = 0.003785 E = 2840 MPa	k = 0.003517 E = 3480 MPa
AC14 high fatigue base course	k = 0.007828 E = 650 MPa	k = 0.006682 E = 1010 MPa	k = 0.005569 E= 1680 MPa	k = 0.005175 E= 2060 MPa
SMA10 wearing course	k = 0.009187 E = 610 MPa	k = 0.007843 E = 950 MPa	k = 0.006537 E = 1570 MPa	k = 0.006073 E = 1930 MPa
OGPA 7/ OGPA10 wearing course	Not used.	Not used.	k not modelled E = 500 MPa	k not modelled E = 500 MPa
DG7 wearing course	Should not be considered in the modelling for pavement design.			

Pls note that AC10 and AC14 surfacing do not meet NZTA T10 texture depth requirements for 70kph design speed and therefore other safety devices may be considered or high friction surfacing to be provided.

5.1.4.4 Catalogue Design for Structural Asphalt

Structural asphalt pavement designs for design CBR values 2%-3%, greater than or equal to 3.5% and CBR greater than 5% for both 10kph and 40kph sites are given in Table 20, Table 21 and Table 22 respectively for traffic loading from only 2 different traffic loading categories - up to 2.5MESA and 2.5 to 5MESA.

Construction traffic and early loading from development traffic must be considered in the design to ensure that the early trafficking will not adversely affect the achieved life of the pavement. In particular, the first lift of the asphalt basecourse should not be trafficked except by the traffic necessary to place the next lift.

If a period of traffic operation on partially completed asphalt is required, this will require bespoke design with the accumulated damage from the construction period calculated and added to the operational period damage to determine the appropriate asphalt thicknesses.

It is important to note that appropriate presumptive design vertical modulus of normal basecourse under the asphalt concrete should not be more than what has been suggested under Table 6.4 of Austroads AGPT-2.

Table 20: Catalogue Structural Asphalt Designs for Subgrade CBR of 2% to 3%

Design traffic (DESA)	Up to 2.5×10^5		From 2.5×10^5 to 5×10^6	
Layer Details / Design Speed	**10kph**	**40kph**	**10kph**	**40kph**
Asphalt wearing course M01-A binder grade	AC14 PG64 H - 55mm		AC14 PG64 H - 55mm	
Tack coat [1]	0.30 to 0.4 litre/m^2 residual binder			
Intermediate course M01-A binder grade	AC14 80mm PG64 H	AC14 65mm PG64 H	100mm AC20 PG64 H	85mm AC20 PG64 H
High Fatigue lower basecourse M01-A binder grade	AC14HF – 60mm PG64S		AC14HF – 60mm PG64S	
Membrane seal	Grade 4 chipseal, 1.4 litre/m^2 residual binder			
Minimum AT AP40 subbase	200mm		200mm	
Subgrade Improvement layer[2]	300mm			
Total minimum thickness (mm)	695	680	710	690

1. Tack coat only required if asphalt intermediate course is opened to traffic before surfacing laid.
2. Subgrade improvement layer can be:
   i. Chemically stabilised in-situ or imported subgrade using lime and/or cement (laboratory soaked CBR testing required to determine dosage to achieve minimum laboratory soaked CBR of 15%.
   ii. Mechanically stabilised by importing black sand, encased in Class C geotextile.

In the event a subgrade of CBR <2% is found on site, natural material or resulting from construction activities, a bescope pavement design is to be carried out with additional subgrade improvement layer. Use the CBR 2% designs in Table 20 with a black sand subgrade improvement layer with thick wrapped in Class C geotextile to be confirmed with design analysis.

Table 21: Catalogue Structural Asphalt Pavement Designs for Subgrade with CBR ≥ 3.5%

Design traffic (DESA)	Up to 2.5×10^5		From 2.5×10^5 to 5×10^6	
Layer Details / Design Speed	**10kph**	**40kph**	**10kph**	**40kph**
Asphalt wearing course M01-A binder grade	AC14 PG64 H - 55mm		AC14 PG64 H - 55mm	
Tack coat [1]	0.30 to 0.4 litre/m² residual binder			
Intermediate course M01-A binder grade	AC20 90mm PG64 H	AC20 75mm PG64 H	AC20 105mm PG64 H	AC20 85mm PG64 H
High Fatigue lower base course M01-A binder grade	AC14HF – 60mm PG64 S		AC14HF – 60mm PG64 S	
Membrane seal	Grade 4 chipseal, 1.4 litre/m² residual binder			
Minimum AT AP40 subbase	200mm		200mm	
Subgrade Improvement Layer[3]	Class C geotextile			
Total minimum thickness (mm)	405	390	420	400

1. Tack coat only required if asphalt intermediate course is opened to traffic before surfacing laid

2. Class C geotextile separation layer to stop fines migration from subgrade for strength less than CBR 4%

In the event a subgrade of < CBR 3.5% is found on site, natural material, or resulting from construction activities, a contingency plan using the CBR 2 to 3% designs in Table 20 shall be shown on the Drawings.

Table 22: Catalogue Structural Asphalt Pavement Designs for Subgrade with CBR ≥ 5%

Design traffic (DESA)	Up to 2.5×10^5		From 2.5×10^5 to 5×10^6	
Layer Details / Design Speed	**10kph**	**40kph**	**10kph**	**40kph**
Asphalt wearing course M01-A binder grade	AC14 PG64 H - 55mm		AC14 PG64 H - 55mm	
Tack coat [1]	0.30 to 0.4 litre/m^2 residual binder			
Intermediate course M01-A binder grade	AC20 75mm PG64 H	AC14 60mm PG64 H	AC20 90mm PG64 H	AC20 80mm PG64 H
High Fatigue lower course M01-A binder grade	AC14HF – 60mm PG64S		AC14HF – 60mm PG64S	
Membrane seal	Grade 4 chipseal, 1.4 litre/m^2 residual binder			
Minimum AP40 subbase	200mm		200mm	
Total minimum thickness (mm)	390	375	405	395

1. Tack coat only required if asphalt intermediate course is opened to traffic before surfacing laid

In the event a subgrade of < CBR 5% is found on site, natural material, or resulting from construction activities, a contingency plan using the CBR 3.5% designs in Table 21 shall be shown on the Drawings.

5.1.5 Concrete Pavements

The below design requirements shall be adopted under catalogue pavement designs for the concrete pavements especially for vehicle crossing, raised safety platform as traffic calming device, carpark adjacent and adjoining road carriageway, and roundabout and intersections. TDM Standard drawings can be referred.

If the existing subgrade CBR is less than 3.5%, then undercut and hardfill to be carried out as per subgrade improvement standard practice.

It is advised to use lapped DH12 trimmer bars 1.5m length at 300mm centre on all four corners or around any opening / perforations such as manholes or service chamber. Trimmer bars should not cross joint locations.

Table 23: Concrete Pavement Thickness for Subgrade CBR 3.5% or Higher

Description	Minimum thickness (mm)	28days crushing strength	Subbase Course Min. Thickness (mm)	Reinforcement Steel
Residential vehicle crossing (up to 4 dwellings)	150	20 MPa	150mm AT AP40	Grade500 665 or SL72 or SE72 mesh centrally placed.
Commercial vehicle crossing (more than 4 dwellings)	200	25 MPa	200mm AT AP40	Grade 500 661 or SL92 mesh centrally placed.
Raised safety platform (Speed Tables) (on existing or new road	Platform – 300 and Toe-slab - 200 (minimum)	40 MPa	150mm ATAP 40 and 30mm AP20 on minimum 200mm ATAP40 existing or new on subgrade with CBR >=3.5	Two layers 663 or SL82 or SE82 Grade 500 mesh. Use 1 layer of sleeved plate dowels centrally placed at formed joints.

Description	Minimum thickness (mm)	28days crushing strength	Subbase Course Min. Thickness (mm)	Reinforcement Steel
On-road carpark adjoining or adjacent to the carriageway.	200	25 MPa	200mm AT AP40 Subgrade CBR > 3.5	Grade 500 661 or SL92 or SE92 mesh centrally placed. Jointing as per Note[1] below.
	180	36MPa		
Roundabout (trucking apron traffic area)	240	40 MPa	200mm AT AP40 Subgrade CBR > 3.5	Either continually reinforce with Grade 500 SE92 at top and bottom 2 layers –75mm cover on bottom and 50mm on top
Roundabout centre island (non-traffic area)	180	25 MPa	200mm AT AP40 Subgrade CBR > 3.5	Continually reinforce with Grade 500 SE92 or 661 mesh centrally placed

1) Sawn 3mm wide joints to minimum 1/4 of the concrete depth (50mm of 200mm concrete) at maximum 4.2m intervals with 2 of every 3 mesh bars crossing proposed joint removed for 75mm either side of joint. Mark sawcut locations before pouring. Expansion joints 10mm compressible foam with 10mm x10mm sealant at maximum 15m intervals. Depending on the length of the concrete pavement and type of the proposed loading, dowels for transfer of vertical loas may be required.

It is advised to submit a bespoke design for concrete roundabout and intersection pavement using Austroads AGPT-2 and NSW Roads and Maritime design standards for reinforcement layout and jointing details. AT has previously reviewed and accepted design using continually reinforced concrete D16 and D12 and fibre reinforced concrete using Dramix steel fibres. However, finished surface on all concrete pavement to have as per Section 7.3 either hard broom U5 / U6 or exposed aggregate finished surface to comply with the minimum texture and NZTA T10 skid resistance requirements.

5.1.6 Subsurface Drainage for Pavement

Subsurface drainage in the form of subsoil trenches and pipes shall be provided as per AT TDM Kerb & Channel Standard Drawing KC0007 and as per Section 2.6. Design for subsurface drainage shall be shown on the pavement or stormwater Drawings, showing minimum grades, direction of flow and discharge or connection point. Discharge invert levels must be above the top of the outlet pipe at catchpits. Levels to be shown on Drawings confirming this. All trenches backfill material shall be TNZ F/2 filter material, filter sock on pipe. Maximum lengths before flushing eyes with surface cover access installed shall be 50m. Subsoil starts and end needs to be either connected to a stormwater structure for cleaning and flushing or start at a flushing eye with a smooth bore uPVC or HDPE pipe at a 45degree angle matching the subsoil pipe diameter for inspection and cleaning. Flushing eye pipes shall finish with a screw cap for inspection and flushing. The screw cap end shall be in a cast iron surface box and cover at a suitable location in the berm area.

At outlets which discharge to embankments or culvert headwalls/wingwalls include in the design for provision of appropriate industry approved grille to prevent vermin nesting.

Raising levels above standard depths may be required in special circumstances, such as where receiving outfall cannot be lowered to achieve discharge, or to avoid subsoil trenches being located within expansive soil subgrades (soaked CBR swell >/= 2.5%).

6.0 BESPOKE PAVEMENT DESIGN

A bespoke pavement design requires the determination of the design subgrade CBR and the design traffic and then using these parameters to model, using appropriate material parameters, the pavement performance based on mechanistic modelling. Bespoke pavement designs allow for optimisation of the pavement structure. Bespoke pavement design shall offer some efficiencies compared to the catalogue designs for the M1 and M2 category or local and collector roads.

Greenfield and rehabilitation pavement design on movement - M3 category – primary / arterial collector and secondary / district collector roads including those with bus routes or roads with very high HCVs volume > 500 per day require a full bescope pavement design following the latest versions of NZ Guide to Pavement Structural Design (NZTA 2018), the NZ Guide to Pavement Evaluation and Treatment Design (NZTA 2018) and Austroads Guide to Pavement Technology Part 2 and Part 5.

Supplementary design and material characterisation requirements are included below:

- Due to the high movements and place value P2/M3 and P3/M3 sites require an economic analysis of the proposed treatments as detailed in the Monetised Benefits and Cost Manual (Waka Kotahi (2021))[3].
- Design traffic shall be calculated in accordance with Section 4.0.
- For the movement classifications M2 and M1 the design can follow the catalogue design approached detailed in Section 5.0, however bespoke pavement designs are also permitted.

The design shall consider critical underground assets identified during the investigation phase, and the design consider how best to protect these assets.

6.1 DESIGN PROCESS

Following sections from 6.1.1 to 6.1.4 applies to primarily for reconstruction and road widening of existing carriageway section for pavement renewal including any intersection or roundabout development for new road connection. In addition, these sections also ate applicable to pavement rehabilitation, and capital or major transport project being implemented within AT network.

6.1.1 Desk Top Study

A desk top study shall be conducted as detailed in the latest version of the NZ Guide to Pavement Evaluation and Treatment Design latest version published by NZTA.

Determine the design traffic as per Section 4.0 of this document. All traffic information must be up to date and may require an assessment of the potential for future land development. The designer may carry out traffic counts for sites where no traffic data is available from Auckland Transport. All new traffic count information must be uploaded into RAMM.

If a traffic model is not available, then the traffic growth rate should be estimated from the historic data. As a last option, a default growth rate of 3% can be adopted, however, adoption of this value will only be accepted if it can be demonstrated that a higher value is not appropriate.

The desk top study should look at the available RAMM pavement and FWD data and identify what additional data should be collected. The desk top study should inform the observations necessary on the site walk over.

[3] https://www.nzta.govt.nz/assets/resources/monetised-benefits-and-costs-manual/Monetised-benefits-and-costs-manual.pdf

6.1.2 Detailed Site Walk-Over

For an existing pavement review or design, a site walk over should be conducted. For a rehabilitation design the approach should follow the process detailed in the latest version of NZ Guide to Pavement Evaluation and Treatment Design, in particular, a thorough defect map should be created to benchmark the pavement condition. Designers are encouraged to identify factors that will impact on pavement performance, in particular, areas where cut or fill will be required, and any drainage paths identified. Preliminary locations for test pits should be identified with the final locations being determined after the Falling Weight Deflectometer and RAMM data are analysed.

6.1.3 Falling Weight Deflectometer Testing

For rehabilitations and road widening projects, the designer must prepare a programme of Falling Weight Deflectometer (FWD) test sites and arrange for all testing, and analysis of data including pavement modelling.

Testing shall be carried out at maximum 20 m intervals in the outer wheel path of each lane. At intersections testing shall be at 10 metre spacings. Smaller sites, less than 300m length, shall require testing at 10m intervals in each lane. All lanes in both directions must be tested unless otherwise agreed prior with the Auckland Transport representative. All FWD testing must be carried out in accordance with ASTM D4695-903 (2020): Standard Guide for General Pavement Deflection Measurements.

Preferably, FWD is undertaken prior to intrusive testing, so that the FWD results can be used to inform appropriate locations for test pits, sampling, or additional assessments. FWD results should be no older than 12 months for M2/M3 roads and no older than 24 months for M1 roads. Older FWD results may not reflect the state of the pavement and subgrade and affect the accuracy of the design.

Traffic Control for the FWD testing is to follow a traffic management plan approved by Auckland Transport.

6.1.3.1 Modelling with Back Analysis

If the intention is to retain existing pavement materials the designer shall create existing pavement models after test pits have been excavated to analyse layer and subgrade strengths. The pavement models will iterate the strength of layers to replicate the FWD deflections and deflection curvature at test pit locations. A summary analysis of deflections will indicate whether the site should be sub-sectioned into more than one existing pavement model. The aim of modelling is to create representative existing pavement models that reflect, depending on design reliability adopted, the 90^{th} or 95^{th} percentile deflections measured by FWD.

The designer may choose to convert the FWD data into Benkelman Beam deflection results (which are higher) before undertaking back analysis. The formula to be used for this was determined by Geosolve and is:

Benkelman Beam / FWD Ratio = 1.1 for deflections up to 1mm

Benkelman Beam / FWD Ratio = 1.1 x (FWD deflection in mm)$^{0.4}$ for deflections > 1mm

Model the Benkelman Beam with 580 kPa tyre pressure and radius of contact 105.06 mm.

FWD providers do provide back analysis strengths of pavement layers, however this is based on isotropic material parameters, although unbound and modified granular layers are considered anisotropic. Furthermore, unless the back analysis has utilised actual pavement layer thicknesses based on the test pit information, it will be much less accurate than modelling the pavement using CIRCLY or AUSTPADS back analysis with the correct layer thicknesses and utilising anisotropic materials.

6.1.3.2 Subgrade Depth for Pavement Modelling

It is noted that pavement depth is generally influence by the subgrade depth up to 2m where the subgrade strength is gradually improved and increased or remained similar subgrade strength unless there are underlying peat soil. Pavement modelling for subgrade is considered as semi-infinity depth and does not automatically limit subgrade depths, and therefore the designer may require to model the subgrade layers if there are likely weaker underlying subgrade layers, FWD analysis and engineering analysis of other information is used as a tool for identifying layer strengths and therefore mode of failure, and pavement treatment options. All results are to be analysed and commented on. The outputs of the FWD testing are to be included with the design report. Where possible homogeneous sections of the pavement should be identified. It is advised to present the results in terms of RAMM carriageway chainages.

6.1.4 Site investigation

A site investigation should be conducted following the processes detailed in the latest version of the NZ Guide to Pavement Evaluation and Treatment Design. Test pits should be appropriately spaced, typically every 100 metres, but in the context of rehabilitation the test pits could be located at points of interest, such as areas identified with high FWD deflections or areas with high level of defects. Visual inspection to check any pavement defects should be carried out to come up with a typical pavement condition. For road widening hand auger testing may be conducted following confirmation of clearance to the utility services.

It is the designer's responsibility to ensure that the necessary information associated with risk is identified and collected for appropriate pavement design.

6.1.4.1 Location of Utility Services

For pavement rehabilitation and upgrade projects, utility plans and written permits for proposed works at each site are to be obtained by the designer who should advise all utility operators of proposed works and to sketch all services on a copy of the base plan. Significant services must be clearly noted, these include, for example, high pressure gas, water, high / low voltage power, fibre-optic cables, telecom cables, main or trunk lines owned by Watercare etc.

The designer must arrange for each utility service authority as part of the design process, to attend the site and mark out the location of their services prior to verification by either the use of ground penetrating radar or utility pilot trenches, or a combination of both methods. For the initial design phase, ground penetrating radar may be used to indicate service locations and then, in subsequent design phases, the locations must be confirmed with trenching.

The information and outputs regarding the location of utility services is to be included with the design report. The report should identify how utilities will be protected during construction. Design provisions are to be made where there are shallow utility services. For example, the treatment of concrete lined steel pipes will require liaison with the utility company to agree on how to protect the pipe and may require relocation, prohibition of vibratory compaction above, or a concrete cap to minimise stresses.

6.1.4.2 Ground Penetrating Radar (GPR)

The designer is to arrange for ground penetrating radar to determine the indicative location and depths of existing concrete slabs, concrete capping of underground utilities and all active and abandoned underground utilities. The GPR results must be accurately surveyed, and the results presented clearly in the design drawings. The designer is expected to have reviewed the GPR results to ensure appropriate design solutions, risk management, safety in design practices and practical contingencies. Furthermore, the designer is expected to use the GPR results to plan and schedule utility or concrete slab pilot trenching. Where existing embedded concrete slabs are to remain within the road carriageway, location, depth and condition information must be recorded within the RAMM database if not already recorded.

The output of the ground penetrating radar study is to be included in the design report.

6.1.4.3 Test Pits and Utility Pilot Trenching

For reconstruction of the existing pavement, the designer following review of the GPR results is to identify shallow services and is to arrange for test pits and utility pilot trenching over critical services in strategic positions highlighted on the base plan and agreed with the Client. Initially ground penetrating radar can be used to locate and estimate the depth of utilities but the actual locations shall be confirmed through test pitting or hydro excavation.

The test pit/utility pilot trenching is to be logged by the designer to NZ Geotechnical Standards and the test pit photos provided within the report must be in colour. High-definition digital photographs must be undertaken with a tape measure in the hole indicating depth and the quality of the samples adjacent. All material testing is to be carried out by IANZ certified technician.

Liaison and necessary permits are the responsibility of the designer and are to be obtained and held on-site as required (i.e. Corridor Access Request and Utilities Approval). Auckland Transport approval must also be obtained to undertake intrusive pavement testing within existing carriageways. Approval decisions are based on the need to obtain carriageway related information for design, funding commitment for full pavement reconstruction and inability to obtain information outside the carriageway.

a) For the subgrade:

- Sufficient and representative samples (for instance no less than three samples per homogeneous section), must be obtained from each site for determination of natural soaked subgrade strength and CBR swell values.
- Shear vane peak and remoulded strength testing starting at the top of the subgrade formation level in each test pit or at representative spacings for greenfield sites.
- A Scala penetrometer test must be started at the top of the subgrade formation levels (or above) and continue to a depth of at least 1.5m below the design subgrade level. Scala testing should be performed in each test pit or at representative spacings for greenfield sites. A refusal result should be retested in the pit base to ensure it is not an isolated obstruction in the subgrade causing the refusal.
- Other subgrade testing methods, such as reactivity testing, and in situ moisture content (laboratory) may be required to confirm the cause of failure or to verify the design assumptions.

b) For the pavement layers:

- Natural soaked granular pavement CBRs and, if required for the design, two modified soaked CBR tests of each distinct layer.
- Other testing such as Atterberg Limit test, plasticity index, particle size distribution test, unconfined compressive test and indirect tensile test may need to be considered if required to confirm the design assumptions.

Where investigation is undertaken on an existing pavement location and spacing of the test pit shall have prior approval from the Auckland Transport representative and Corridor Access Request (CAR). Each test pit must be saw cut square prior to excavation and minimum dimensions are 400mm by 400mm. The excavated depth is to the maximum of either the existing subgrade level or a depth of 800mm.

Reinstatement of the test pits must be as specified in the Utility Operators Code of Practice and achieve a life of at least 12 months. Such reinstatement in the carriageway area to be as per minimum standard prescribed under approved CAR. Generally, backfilled materials shall have generally AP40 granular basecourse materials compacted in 200mm layers to minimum CiV 35 when tested using Clegg Hammer underneath the surfacing layer and 40mm DG10 asphalt concrete or approved alternative surface following a heavy tack coat.

All test results including interpretation of subgrade assessment for pavement design should be discussed and reported within the pavement design report.

6.1.5 Laboratory testing

All laboratory testing is to be carried out by an approved IANZ (International Accredited New Zealand) certified Civil Engineering Laboratory.

All testing shall be the responsibility of the designer. Testing must provide enough information to establish the condition of the various existing pavement layers and materials and sufficient to support and justify any design option chosen. Guidance on laboratory testing is detailed in the latest version of NZ Guide to Pavement Evaluation and Treatment Design.

The outputs of all laboratory test results and related interpretation to be used as design parameters are to be included with the design report.

6.1.6 Topographical Survey and Preliminary Design Drawings

Topographical survey must be carried out for both existing, brownfield or greenfield sites as per approved by the Auckland Transport representative or as deemed necessary of the catchment or extent of work. For all existing roads, it is necessary to ensure adequate longitudinal and cross section shape and gradient on road surface and adequate stormwater drainage.

The topographical survey must record details including road and kerb levels, street furniture, service covers, drainage features, drip lines, legal boundary, vertical profile at vehicle crossings, and other significant information.

Survey accuracy must be ±10mm for reduced levels and ±20mm for horizontal locations. The survey must be presented in AutoCAD format and must comprise a layout plan showing all existing features, longitudinal sections, along the road centre lines and kerb lines, and cross sections at no more than 10m intervals. The topographical survey plan must include contours at 100mm intervals depending on the road gradient. For sites with less than 2% gradient, closer spacing intervals are required. Appropriate Temporary Traffic Management for the topographical survey must be included and obtained approval from the local authority.

All relevant geometric and stormwater design drawings are to be included with the design report.

6.2 PAVEMENT REHABILITATION

Pavement rehabilitation or strengthening design shall follow the latest version of the NZ Guide to Pavement Evaluation and Treatment Design, the NZ Guide to Pavement Structural Design and Austroads AGPT-2. The design process should follow the same process as detailed in Section 6.1 of this TDM.

The pavement models developed should demonstrate theoretical deflections and curvatures that meet the requirements of Table 25.

In addition, keying or joining the rehabilitated pavement into the existing pavement is a critical design feature, the principles detailed in Section 2.8 are applicable to constructing a suitable interface.

An inspection and test plan like sample provided in Appendix A should be developed to demonstrate that the design assumptions are met during material supply and construction.

6.3 MODELLING CHARACTERISATION

For all bespoke pavement designs the pavement materials should be characterised in the pavement models as detailed in the following sub-sections.

6.3.1 Subgrade Improvement Layers

Subgrade improvement layers shall have maximum values as follows, refer below for sub-layering rules where lesser values might apply:

- For imported black sand wrapped in geotextile maximum CBR 8% (presumptive modulus 80 MPa)
- Imported or site won subgrade improvement material, determined by soaked CBR testing, or CBR 10% (presumptive modulus 100 MPa) whichever is the lesser
- Subgrade chemical stabilisation, determined by soaked CBR testing, or CBR 10% (presumptive modulus 100 MPa) whichever is the lesser
- Subgrade mechanical stabilisation CBR 8% (presumptive modulus 80 MPa)

Subgrade improvement layers shall be modelled according to Section 8.2.2 Austroads AGPT-2. This includes imported or site won cohesive selected subgrade improvement layers, black sand wrapped in Class C geotextile, and chemical or mechanically stabilised existing subgrade materials. Applicable AGPT Part 2 equation is:

$$E_{v\ top\ sublayer} = E_{v\ underlying\ material} \times 2^{(thickness\ of\ each\ selected\ subgrade\ or\ stabilised\ subgrade\ layer/150)}$$

The ratio of modulii of adjacent sublayers is given by:

$$R = \left[\frac{E_{v\ material\ top\ sublayer}}{E_{v\ underlying\ material}}\right]^{\frac{1}{5}}$$

Only crushed rock or recycled crushed concrete shall be modelled in accordance with Section 8.2.3 Austroads AGPT-2; and the applicable equation is:

$$E_{v\ top\ granular\ sublayer} = E_{v\ underlying\ material} \times 2^{(total\ granular\ thickness/125)}$$

The ratio of modulii of adjacent sublayers is given by:

$$R = \left[\frac{E_{v\ top\ granular\ sublayer}}{E_{v\ underlying\ material}}\right]^{\frac{1}{5}}$$

6.3.2 Unbound or Modified Granular Materials

The design thickness of unbound or modified granular material (and any subgrade improvement layer included), over the in-situ subgrade, is determined using the empirical design chart given in Figure 8.4 of Austroads AGPT-2. Note that Austroads Figure 8.4 can be used to pavements with design traffic loading up to 1 x 10^6 ESA for urban roads and up to 5 x 10^6 ESA for rural road as per Table 9 in Section 4.0.

Note that CIRCLY or AUSTPADS modelling is not to be used for determining the thickness for subgrade rutting failure of unbound or modified granular pavements. Austroads Figure 8.4 can be used for this purpose. CIRCLY or AUSTPADS modelling shall be used to check Table 25 deflection and deflection curvature requirements (in the case where asphalt surfacing will be applied) on the top of the basecourse layer are met.

The total thickness of material over the in-situ or virgin ground subgrade may be made up of the following materials:

- Unbound granular base and subbase courses.
- Imported or site won selected subgrade improvement layer materials.
- Chemically stabilised subgrade material, provided that the material has sufficient chemical content to ensure design properties are achieved for long-term performance. If the amount of lime is insufficient to achieve long-term strength, no allowance should be made for the increase in subgrade CBR due to stabilisation.
- Mechanically stabilised subgrade layers where granular materials are compacted into the subgrade material to create a stronger subgrade.

The composition of the unbound pavement is made up by providing sufficient cover over the in-situ subgrade and the selected subgrade improvement layer or stabilised subgrade layer.

For granular pavements with thin surfacing it is necessary to provide a minimum thickness of a suitable basecourse material. This minimum thickness of basecourse, complying with AT TDM Specification for Supply of Aggregates, shall be not less than 125mm for constructability to aid compaction and level control. Single layer thickness up to 200mm for basecourse can be considered. Refer to NZTA M4 notes for Class of M4 basecourse appropriate to design traffic ESA level.

For granular pavements with thin surfacing, it is necessary to provide a minimum thickness of suitable basecourse material. This minimum thickness of basecourse complying with AT TDM Aggregate Specification using NZTA M4 AP40 Class 2 aggregates for roads Arterial and Collector roads including those with major Buses routes to be 150mm and AT AP40 or NZTA M4 Class 2 for local roads shall be not less than 125mm for constructability to aid compaction and level control. Single layer thickness up to 200mm for basecourse can be considered.

When checking deflections on top of unbound or modified basecourse materials, these should be modelled as having a maximum presumptive resilient modulus 350MPa. Similarly, a presumptive modulus for unmodified subbase course AP65 of 150 MPa is appropriate. These stiffnesses have been adopted for design deflection modelling to match back analysed stiffnesses in Figure 8.4 of Austroads AGPT-2. They represent the long-term strength of the pavement aggregates after initial higher strengths are reduced by repeated loading and fines migration, as does Figure 8.4 (which is based on the long-term rutting performance of many pavements in Australia).

The increased stiffness from cement modification has limited duration, therefore, the pavement model should use the unmodified stiffness.

Existing granular materials being used in the design may have lower stiffness values, and these shall be determined from FWD data back analysis as outlined above. Relevant Repeated Load Triaxial (RLT) results may be carried out to provide the stiffness of existing granular materials and rutting resistance for use in the pavement modelling, with the stiffness being calculated at appropriate stresses.

Granular materials constructed on a stabilised subbase do not need to be sublayered as per Austroads AGPT-2.

In all cases however, other modulus limits shall be considered, including stress dependency limits as per the Tables 6.4 and 6.5, and sub-layering modulus limits as per equations 39, 40, 41 and 42 respectively of Austroads AGPT-2. In particular, the suggested vertical modulus values in Tables 6.4 and 6.5 from Austroads should be considered for granular materials placed beneath bound materials.

In pavement modelling, all base and subbase course comprising crushed rock shall be sublayered in accordance with the Austroads AGPT-2 sub-layering procedure. The design vertical modulus value in the vertical direction at the top of the layer lower subbase layer and top basecourse shall not exceed resilient modulus of 150 MPa and 350 MPa respectively. A Poisson's Ratio value of 0.35 shall be used for all granular material. Granular materials shall be considered as anisotropic with a degree of anisotropy of 2.

Subject to approval by AT, recycled crushed concrete aggregates or naturally occurring gravel pavement materials may be considered as a granular material for use as lower subbase in lieu of crushed rock. However, the maximum design vertical resilient modulus value at the top of a gravel layer of subbase shall not be more than 150 MPa.

6.3.3 Cement Modified

A cement modified material shall have a dry Indirect Tensile Strength (ITS) between 150 to 350 kPa or a wet ITS between 100 to 300 kPa when tested in accordance with NZTA T19 (2020). Cement modification is used to improve the strength of marginal quality basecourse material.

Cement modification of the basecourse shall not assume any long-term increase in the basecourse stiffness and is limited to 350MPa vertical modulus for modelling, sub-layering, and anisotropic characteristics. As for unbound material, stress dependency limitations (when overlying stiffer materials are present) and modulus gain limitations from layers below will define what top modulus can be achieved for design.

Cement modified basecourses shall have particular care in preparing the surface for sealing to prevent a cake of cement laitance forming, and a prime coat applied prior to placing the membrane or a chipseal.

6.3.4 Foamed Bitumen

Foamed bitumen stabilisation may be performed on existing granular materials for rehabilitations or on new granular materials for new pavement construction. Foamed bitumen pavements shall be modelled as detailed in the latest version of NZ Guide to Pavement Evaluation and Treatment Design published by NZTA with the mix design requiring a phase two modulus greater than 800 MPa.

Note that mix designs with a phase two modulus greater than 1200 MPa may be at risk of block cracking and therefore the consequences should be identified and managed, potentially through revision of the mix design.

6.3.5 Cement Stabilised Subbase

A cement stabilised subbase material shall have a dry Indirect Tensile Strength (ITS) greater than 500 kPa or a wet ITS greater than 450 kPa when tested according to NZTA T19 (2020).

Cement stabilisation of the subbase will likely require in-situ cement content between 3 and 5% depending on the quality of the subbase material used. If premixed cement is used, compaction of the pavement must be completed within 2 hours of the mixing of the cement and water. Trafficking of the subbase shall be minimised to allow only traffic necessary to construct the subbase course and the overlying basecourse.

Sufficient material shall be under the cement bound subbase:

- To ensure an anvil suitable for compaction of the cemented layer is present, and
- For in-situ stabilisation a buffer layer minimum 100 mm thickness under the depth of stabilisation to ensure improved subgrade materials with lime stabilisation will not be hoed into subbase material.

Cement stabilised subbase shall be modelled as a cracked anisotropic material with a modulus of 500 MPa and with no sublayering. However, the curing and trafficking of the subbase should be considered in developing the final adopted parameters.

When in-situ stabilised, the depth of effective stabilisation shall be modelled as 50 less than the actual stabilisation depth. It also recommended a minimum of 100mm subbase course to be available for anvil to carry out in-situ stabilisation of the subbase course.

As detailed in Section 2.9.1, the minimum thickness of unbound basecourse under the kerb and channel is 50mm, and sub-surface water must drain to the pavement drainage system to either open side drain or subsoil drain, therefore, the basecourse thicknesses designed above a cement bound subbase course shall allow for this drainage path.

Where subsoil drains are included the subbase course stabilisation must terminate at the side of the subsoil trench. This allows moisture and sub-surface water within the basecourse layer to reach the subsoil drain, which must connect with unbound aggregate to the basecourse layer.

An unbound basecourse is recommended above the cement bound subbase course, provided the moisture levels can be controlled, and this will likely mean not constructing during the wet months. Where moisture levels are likely to impact the workability and performance of the unbound aggregate, light cement modification may be considered to reduce moisture susceptibility. If required, the cement content when modifying the basecourse must not exceed 1.5% or generate a dry ITS greater than 350 kPa when tested in accordance with NZTA T19.

6.3.6 Lean Mix Concrete

Pavement design using lean mix concrete 10MPa to 20MPa 28day crushing strength as a subbase can only be used where the subgrade is particularly weak, sensitive or to minimise pavement thickness to avoid relocating shallow or critical utility services. Lean mix subbases are also useful for minimising the compaction effort required, compared to a cement stabilised subbase, and thereby protecting underlying utilities and services.

Lean mix concrete subbase course shall be laid on a Class C geotextile to prevent subgrade fine material migrating upward through shrinkage cracks in the material. Lean mix or stronger concrete shall be modelled as detailed in Table 8.4 of Austroads Part 2 (Austroads 2017) as:

- A cracked anisotropic material with a modulus of 500 or 350 depending on the mode of cracking, the cracked layer shall be sublayered and Poisson’s ratio 0.35, or
- If placed with a screed, the lean mix can be modelled as an isotropic 700 MPa material with a Poisson’s Ratio of 0.2 and non-sublayered.

6.3.7 Post-cracking Phase in Cemented Material and Lean-mix Concrete

As per Austroads AGPT-2, typically a thick (≥ 175 mm) asphalt on a cemented material or lean-mix concrete subbase pavement would be modelled by taking account of the fatigue lives of both the cemented layer and the asphalt layer.

To reduce the risk of reflective cracking the pavement should provide a minimum cover equivalent to 175 mm of asphalt over the cemented material or lean-mix concrete. Granular material can be used as cover either solely (i.e. any sprayed seal or thin asphalt surfacing is not considered to be part of the cover), or in conjunction with asphalt, subject to the following criterion:

$$(0.75 \times \text{thickness of granular material cover}) + (\text{thickness of asphalt cover}) \geq 175 \text{ mm}$$

6.3.8 Asphalt

6.3.8.1 Thin Asphalt Surface Modelling

Thin asphalt is defined by Austroads AGPT-2 to have a single layer with a thickness less than 40mm. It should be noted that Austroads fatigue models are not valid for asphalt less than 40mm. Therefore, thin asphalt surfacing less than 40mm thick including open graded asphalt should not be considered as pavement layer in the structural design.

The design curvature values in Table 25 are intended to keep the strain levels low and ensure the performance of the asphalt wearing course.

Asphalt wearing courses have the following minimum thicknesses in accordance with NZTA M10:

- DG7 mix with a minimum of 30mm can be used for off-road dedicated carparks and shared paths.
- AC10 or SMA10, DG10 have a minimum lift thickness of 40mm.
- AC14 has a minimum lift thickness of 55mm.

AC14 at minimum 55mm can be modelled for fatigue layer.

Designers are reminded that the expected life of a wearing course is approximately 12.5 years for M3/M2 arterial and collector road pavements and 15 years for M1 or local road pavements. Therefore, for a design life of 25 years, a cumulative distribution function (CDF) that indicates these expected lives of the wearing course will be achieved may be greater than 1 but should be less than 2. No pavement surfacing or wearing course should be designed for a design life less than 10 years.

Please note that as per NZ Guide to Pavement Structural Design Guidelines, the modulus of the asphalt surfacing is assumed to have the same as the FBS, not the actual asphalt modulus. The designers are advised to check when modelling thin asphalt (40mm or greater thickness) that the actual asphalt in service modulus and fatigue parameters is used or assumed values when the mix supplier is not known.

Bitumen binder shall be selected in accordance with NZTA M01-A specification. The addition of polymer shall not be used as a justification for reducing asphalt thickness. If a bespoke k and b value for use in the fatigue equation design process is determined by the Contractor for a particular mix design, in accordance with AGPT/T274, then these values may be used for asphalt fatigue modelling.

6.3.8.2 Structural Asphalt

Structural asphalt pavements shall be modelled with at least one layer of basecourse asphalt and an asphalt surfacing. A high fatigue asphalt layer can be used below an intermediate asphalt layer to reduce the total asphalt thickness when unbound granular base is used. For structural asphalt on unbound or cement treated subbase course a minimum thickness of 170mm of AP65 subbase aggregate or 150mm AP40 basecourse aggregate shall be included.

All asphalts used in pavement construction should be approved by Auckland Transport and registered on their list of approved asphaltic concrete mix designs.

Four traffic categories of Light, Medium, Heavy and Very Heavy have been chosen and a guide to the selection is shown in Table 3.2 of NZTA M10 Notes. The relevant traffic category should be nominated in the detailed design drawings. Similarly, the nominal size can be determined as a function of the layer thickness, or the layer thickness selected based on the nominal size required for a particular application. A guide to selection of layer thickness and nominal size is shown in Table 3.3 and Table 3.4. of NZTA M10 Notes.

6.3.8.3 Design Speed

Posted design speed for local roads to be designed as 40 kph for mid-block and all intersection and roundabouts, raised safety platform and any low speed designed area to 10 kph as detailed in Section 5.1.4.2.

All bus station / terminals including bus depots / lays-over area are to be designed to 10kph or for static load condition in parking areas, with a speed adjustment factor of 0.3 as per Austroads AGPT-2 Figure 6.10. This is the ratio modulus at vehicle in service speed to modulus from the standard indirect tensile test (40ms rise time).

6.3.8.4 Air Voids & Operating Temperature

Based on a construction tolerance allowance, and experience with paving, asphalt shall be modelled assuming that the in-service air voids are 1% higher than the design air voids for the mix being considered. For simplicity, it is suggested to use construction air void 5% for the sake of deriving adjusted presumptive design modulus.

AT has considered the effects from Climate Change for the temperature adjustment factor. Due to potential increasing temperatures, and higher temperatures associated with increased high-density housing in the urban environment, the Weighted Mean Annual Pavement Temperature (WMAPT) or operating temperature to be used for design of maintenance and rehabilitation is 24°C and for new pavements is 25°C.

Use AGPT Part 2 Equation 23 to calculate the ratio of the field modulus at the in-service temperature (WMAPT – 24deg C) to the modulus at the laboratory test temperature (25 °C).

6.3.8.5 Design Parameters

Only Auckland Transport accepted asphalt mixes are permitted for use. All submitted mix designs including Stone Mastic Asphalt (SMA) by asphalt suppliers are reviewed and validated annually. Open Graded Porous Asphalt (OGPA) mixes are not currently within the approved list of mixes. For M3 / arterial and primary collector road carriageways, project specific approval is required from the Auckland Transport Asset Management team for their use.

The design life for all asphaltic concrete surfacing must be a minimum of 12.5 years for arterial and collector or M2/M3 roads and 15 years for local or M1 roads.

Auckland Transport annually carries out a review of asphalt mixes including laboratory and field testing of mix design and production. Acceptance letters are issued to the asphalt manufacturing plants upon finding the materials compliance with the relevant standards such as wheel tracking, and the volumetric requirements including successful and satisfactory laydown trials. Auckland Transport has analysed a range of approved mixes from asphalt suppliers and determined appropriate design parameters for use in asphalt fatigue modelling, irrespective of asphalt supplier, for designers to use. Refer Table 14 in Catalogue Designs for the values to be used for Bespoke Designs.

If a designer, through bespoke design, wishes to use alternative asphalt fatigue parameters to those in Table 14, this can be done provided:

1. The mix adopted is an approved mix by Auckland Transport.
2. The mix will be used on the site being designed. Only mixes that are confirmed to be used on the site and clearly identified on the Drawings by asphalt Job Mix Formula designation and date, can be considered for variations to Table 19.
3. The parameters used shall be taken from the Job Mix Formula for the approved mix and utilise:
    a. Effective binder content
    b. ITS modulus adjusted for design speed, temperature and air voids in Section 6.3.8.3 and 6.3.8.4
    c. k value determined from the above with b = 5 in the Shell Fatigue Equation, or
4. If an Auckland Transport approved mix has bespoke k and b values determined following procedures in the latest version of ATM-274 then these may be used, if firstly approved by Auckland Transport.

It is highly recommended that the designer carries out designs using pre-approved asphalt mixes, for all engineering application (EA) application and reviewing process.

Design deflections on top of the layer below the surfacing shall comply with Table 25.

6.3.8.6 Bitumen Binder

The bitumen binder shall be selected using NZTA M01-A for the traffic level and mix type for surfacing or base/ lower layers. The use of binders, which have been modified by the incorporation of polymers (PMBs) can result in some improved fatigue lives when tested in the laboratory. However, due to the variation in polymers available in New Zealand there are currently no fatigue relationships available to confidently estimate the in-service fatigue life of mixes with other than standard grade conventional binders. There is no reference in NZ Guidelines / NZTA standards for savings due to polymer addition, and NZTA M01-A has purposely removed PMB references and has adopted a performance grade approach to asphalt binders. Addition of a polymer is up to the asphalt supplier if needed to meet NZTA M01-A performance requirements.

The addition of polymer shall not be used for adjustment of the design traffic loading or alternatively as a justification for reducing asphalt thickness.

If a bespoke k and b value for use in the fatigue equation design process is determined by the Contractor for a particular mix design, in accordance with AGPT/T274, then these bespoke values may be used for asphalt fatigue modelling. In general, PG64 performing binder grade shall be selected as per Table 2 of NZTA M01-A.

AT advises to adopt the following performance grade binders based on the road category:

Table 24: Performing Binder Grade & Recommended Mixes

Road Use Description	Traffic Volume (ESA)	Traffic Speed (kph)	Binder Grade Category (AASHTO M332)	
			Binder Grade Surface Mixes	Base Mixes
M1 or Local - residential streets / Accessway and Carpark, Cycle Lane and Footpath	$<5X10^5$	All speeds 40 & 10	PG64S DG10, AC10	PG64S AC14 & AC20
M1 / M2 or Local Road and Collector without any bus route	$>5X10^5$ & $<1x10^6$	All speeds 40 & 10	PG64S DG10, AC10	PG64S AC14 & AC20
M2 Secondary Collector with bus service	$>1X10^6$ & $<5x10^6$	>40	PG64H AC10, AC14	PG64H AC14 & AC20
		<=10	PG64V AC10, AC14	PG64H AC14 & AC20
Primary Collector & Arterial with or without bus service	$>5X10^6$ & $<20x10^6$	>40	PG64V AC10, AC14. SMA10	PG64H AC14 & AC20
		<=10	PG64V AC10, AC14, SMA10	PG64H AC14 & AC20
	$>20x10^6$	>40	PG64V AC10, AC14, SMA10	PG64V AC14 & AC20
		<=10	PG64E AC10, AC14 SMA10	PG64V AC14 & AC20
Roundabout / Intersections	$>10x10^6$	<=10	PG64E AC14, SMA10	PG64V AC14 & AC20

Based on the many research, selected Polymer Modified Binder (PMB) asphalt showed reduced rutting and reflective cracking although NZTA M01-A removes the need to specify PMB and, if a PMB is needed to meet M01-A requirements the asphalt supplier is expected to add these to the mix to achieve the performance of binder grade. It is advised to use polymer modified binder asphalt mixes but not limited to surfacing on roundabout and intersections and concrete bridge deck surfacing as per to mitigate rutting / deformation and reflective and fatigue cracking of asphalt. Polymer modified binder needs to go testing as per AASHTO M332 and pass all performance criteria for higher grade performing binder.

All surfacing over concrete decking over bridges to have levelling mixes PG64 S AC7 / AC10 mixes follow by AC10, SMA10 or AC14 PG64 V or better performing grade binder as per NZTA M01-A with polymer modified binder to mitigate the reflective cracking of concrete deck.

Performing grade S, H, V, and E applicable for dense-graded mixes AC and DG grades in M10 and grade V & E are generally applicable for stone mastic asphalt surfacing materials as per NZTA M27 specification.

6.3.9 Rigid Pavement

6.3.9.1 Cast-in Situ Concrete

Transport for New South Wales (TfNSW) is the pre-eminent Australian road authority in rigid pavement technology, having designed, constructed, and maintained most Australian concrete pavements over the last 40 years.

The preference for surfacing a rigid pavement in areas where posted speed is greater than 70 kph is asphalt or other micro-surfacing such as calcined bauxite, provided the risks of reflective cracking are addressed. For concrete in speed zones up to 70 kph a standard hard broom or grooving finished surface or exposed aggregate finish surface can be considered to texture and skid resistance requirements. For further detailed design, it is advised to refer TfNSW Supplement to the Austroads Guide to the Structural Design of Road Pavements (2015). This advice may be extended or superseded by TfNSW Supplement updates and all rigid pavement design should be reported on bescope pavement design including jointing details.

The Austroads design procedure assumes that the base and subbase are debonded. Debonding procedures vary markedly according to both the type of subbase and the type of base and are detailed in TfNSW specifications and Austroads. Appropriate debonding can be critical to the performance of the pavement.

Further information on materials, design and construction aspects is available in the TfNSW QA Roadworks Specifications (for concrete pavements), TfNSW Guides to QA Specifications, and the TfNSW Standard Drawings - Pavement.

Rigid base pavement designs shall generally follow the methodology of Chapter 9 of Austroads AGPT-2 and TfNSW design procedures. The preference for surfacing a rigid pavement is asphalt or micro-surfacing such as calcined bauxite provided the risks of reflective cracking are addressed, however, it may be possible to obtain a departure from Auckland Transport for a concrete surface, provided it can be demonstrated how the long-term skid resistance can be maintained – minimum 15-years before any grooving action is required for minimum surface texture depth.

Jointless continuously reinforced concrete pavement, or bespoke design jointless steel fibre reinforced concrete pavements, with sufficient steel to limit crack widths to 0.25 mm are acceptable if appropriate design methodologies are provided and accepted by Auckland Transport.

For pavement design of bus terminal and carparking areas, static load conditions with appropriate loadings should be considered using various scenarios as provided in the Concrete NZ recommended design analysis.

6.3.9.2 Roller Compacted Concrete

Roller-compacted concrete (RCC) is a type of non-reinforced concrete construction placed with high density paving machine and then compacted with vibratory rollers. It is a dry high-strength concrete pavement material, which is compacted by external rolling rather than internal vibration. While the finished product is essentially concrete pavement, the engineering and construction of the RCC pavement is very different from conventional concrete pavement not requiring large water content. This allows for faster curing time and more cost-effective construction for pavements with little or no formwork or steel reinforcement. Fly ash (FA) is commonly used as a supplementary cementitious material (SCM) in RCC mixes.

The major advantage is that RCC often doesn't require steel reinforcement or formwork, further contributing to its speed and cost-effectiveness as t is made of very low slump concrete with less water. Pavement design and construction to be undertaken.

The most obvious difference between the two is their appearance: RCC cannot be finished or textured like conventional concrete, it has an appearance similar to grey asphaltic concrete. RCC has been in use in the USA for some years but was first introduced to Australia, as a pavement material in 1986. Recently, RCC pavement has been constructed for major roundabouts in Tauranga City and considered to be const effective due to faster construction and curing, and therefore trafficable within 48 hours. Auckland Transport is also undertaking trials on using roller compacted concrete (RCC) pavement, which could be a robust design solution for heavily trafficked electric bus lanes – especially transit routes.

Further details and outline of materials, design and construction characteristics of RCC pavements can be found in Australian Society for Concrete Pavement (ASCP), Australia and Tech Brief FHWA-HIF-16-003 https://www.fhwa.dot.gov/pavement/concrete/pubs/hif16003.pdf and RCC Pavement Council http://rccpavementcouncil.org/what-is-roller-compacted-concrete-pavement/.

6.4 DEFLECTION AND CURVATURE

6.4.1 Deflection Measurements

Pavement deflection measurements can be performed using Falling Weight Deflectometer (FWD) or the Benkelman Beam (BB). Section 6.1.3 provides a method to convert between the FWD and BB deflections.

6.4.2 Pavement Modelling

When modelling theoretical Benkelman Beam deflections, the design parameters as detailed below should be adopted.

- In CIRCLY and AUSTPADS the deflection and curvature function check shall be modelled as a full axle with a tyre pressure of 580 kPa and radius of 105.06mm. This reflects the actual tyre setups on Benkelman Beam trucks to achieve the contact area.
- Subgrade should be modelled as a design modulus, with the layer thickness being semi-infinity when using CIRCLY or AUSTPADS
- If deflections are determined using AUSTPADS, the calculated deflection (in the Load 1 AUSPADS sheet) at X=165mm, Y=0mm and Z=0.1mm depth shall be used as the Benkelman Beam deflection when modelled using a special load case meeting the load requirements with 580 kPa and 105.06mm radius of contact. Designers should note that the AUSTPADS coordinate system datum is different to that used in the CIRCLY software as x=0 is under a tyre, whereas in CIRCLY it is usually between dual tyres.

The Benkelman Beam deflections and curvature values detailed in Table 25: Maximum Benkelman Beam Deflections on top of Basecourse (prior to Surfacing) are the maximum allowable deflections for modelling prior to surfacing with an asphalt wearing course. The curvature is a modelled only value as curvatures cannot be easily field measured with a Benkelman Beam. Curvature values are intended as a design check, they are not construction test targets.

Table 25: Maximum Benkelman Beam Deflections on top of Basecourse (prior to Surfacing)

Road Hierarchy	SARF - Movement Category	One Network Road Classification	Benkelman Beam Deflection (mm) (design & construction)	Curvature Function (mm) (for design of pavement with asphalt surface only)
Arterial & Primary	M3	Arterial	1.0 (maximum)	0.15 (maximum)
Collector (Business)		Primary Collector		
Local (Business)	M2	Primary Collector	1.0 (maximum)	0.15 (maximum)
Collector and or Local with buses		Primary Collector		
Collector without buses	M1/M2	Secondary Collector	1.2 (95th percentile) 1.3 (maximum)	0.18 (maximum)
Local roads and carparks (ESA >= 1x 10^5)	M1	Access	1.4 (95th percentile) 1.5 (maximum)	0.20 (maximum)
Residential street less than 200m length with no exit (ESA < 1x 10^5)				0.22 (maximum)

Measured deflections on basecourse are highly dependent on the moisture content of the pavement. A membrane or chipseal seal may be considered to seal the pavement especially in winter to prevent high moisture content prior to testing, however, the sealing is at the Contractor's risk if subsequent deflection testing fails the requirements. In addition, it is advised to check the performance of the subsoil drainage system.

7.0 SURFACING

7.1 SPRAYSEALING / CHIPSEALING

7.1.1 Chipseal Wearing Course

It is It is essential that the existing Auckland Transport Reseal Guidelines (2013) and Seal Extension Guidelines are read before reading the rest of this Section. These Guidelines are provided in Appendix F of this TDM.

Where a chipseal wearing course is required the detailed seal design must be undertaken by the designer or the contractor generally as per the requirements of the Transport Design Manual (TDM) and the methods outlined in the NZTA publication Chipsealing in New Zealand (Transit 2007). Chipseal should be designed to waterproof the road and provide an appropriate level of skid resistance. The designer/contractor also to submit sealing design for second coat reseal which is needed to achieve the seal life required for sealing on new or rehabilitated pavements.

The first coat seal design, in combination with the second coat seal, should have a combined minimum design life of 10 years on major rural arterials / collector roads (with AADT less than 10,000 roads) and 12 years for local roads. Other than for membrane sealing, the texture requirements for the site may limit treatment options, the proposed treatment should be discussed with Auckland Transport to agree to expected design lives. High stress, high volume, sites may require alternative treatments to chipseal.

7.1.2 Chipseal Design

Unless otherwise specified, sprayed seal treatments shall be designed in accordance with NZTA publication Chipsealing in New Zealand (Transit 2007). The class of aggregate shall be selected in accordance with NZTA M06 specification. Primed surfaces on granular pavements shall be cured prior to applying the final surfacing.

The seal design for sites with insufficient skid resistance must be undertaken as per Appendix A NZTA Maintenance Guidelines for Local Roads (2012). The designer must undertake surface skid resistance testing following the NZTA T10 specification.

The fundamental requirement for new surfacing aggregate type should be the objective of maintaining the skid resistance at or above the threshold level (TL) for the design life of the surfacing. Where this is not cost effective, alternative aggregates may be used. Refer to NZTA T/10: 2013 Notes.

All primer shall be allowed to cure prior to application of the final surfacing. If the use of a proprietary grade of bitumen emulsion primer in lieu of very light cut-back bitumen primer is approved for use, the rate of application of bitumen emulsion shall be such as to deliver an application rate of residual binder as recommended in Table 21 Typical Application Rates. These rates are to be verified on site by the Contractor's seal designer, being adjusted as necessary for site conditions. Polymer Modified Binder (PMB) shall be selected in accordance with Chapter 6 Chip Seal Design unless otherwise specified in the design.

At the time of undertaking the design process on site, the seal designer must look for any tree canopy issues that may hamper chip sealing operations. They will then provide a methodology to prevent damage to tree canopies. Site sketches showing existing pavement features (existing patches/flushed areas/services) areas of traffic stress, location of sand circle tests and large trees must also be included.

7.1.3 Design Traffic

The designer must obtain traffic count data records from the RAMM database, MobileRoad or Auckland Transport traffic count database available from the web site https://at.govt.nz/about-us/reports-publications/traffic-counts. However, it is the responsibility of the Contractor's seal designer to confirm the actual traffic counts for use in the seal / surfacing design. The seal designer may carry out traffic modelling or counts for sites where no traffic data is available from Auckland Transport.

All new traffic count information must be uploaded into RAMM. If the designer proposes an alternative treatment selection, they may forward a written proposal to Auckland Transport. This proposal must be documented with reasons for the alternative and its benefits to the network. Auckland Transport will assess the alternative treatment selection proposed by the designer and will provide recommendations and comments to approval before work starts.

7.1.4 Chipseal Design Report

Auckland Transport must have a minimum of 10 working days to evaluate the seal designs. Work must not begin until the seal designs and treatment selection proposals have been reviewed and comments provided by the Auckland Transport representative. Materials used in chip sealing must comply with the specifications as listed below. All material sampling and testing must be performed by a laboratory which holds either accreditation by International Accreditation New Zealand or registration to ISO Guide 25:1990 for the specified tests, or alternative certification acceptable to Auckland Transport.

The bitumen used must comply with all the requirements of NZTA M01:2022 or the latest NZTA specification for chip seal binder which replaces it.

The sealing chips must comply with the requirements defined in the current edition of the NZTA Specification for Sealing Chips M6 (2019) which sets out the material requirements for five size grades of sealing chip for use on state highways and other heavily trafficked roadways.

The frequency of testing for source and production properties should follow the latest version of the guidelines Quality Assurance of Aggregates for Roads Civil Contractors New Zealand (2019). Synthetic aggregates may be used but require approval from Auckland Transport.

Table 26: Typical Seal Application Rates

Seal Type	Chip Size	Traffic Volume AADT (Both Directions)			
		250 - 500	500-1,000	1,000- 2,500	2,500- 10,000
Emulsion Prime coat		0.50	0.50	0.50	0.50
Membrane Chipseal	Grade 3	1.8	Membranes under asphalt		
	Grade 4	1.4			
	Grade 5	1.1			
Single Coat Chipseal	Grade 2	2.1	2.0	1.8	1.7
	Grade 3	1.8	1.6	1.5	1.5
	Grade 4	1.4	1.3	1.2	1.2
	Grade 5	1.1	1.0	1.0	0.9
	Grade 6	0.9	0.8	0.8	0.8
Two Coat Chipseal	Grade 2 & 4	2.2	2.0	1.9	1.8
	Grade 3 & 5	1.8	1.6	1.5	1.5
	Grade 4 & 6	1.4	1.3	1.2	1.2
	Grade 2 & 4	2.1	2.0	1.8	1.7
Racked in Chipseal	Grade 3 & 5	1.8	1.6	1.5	1.5
	Grade 4 & 6	1.4	1.3	1.2	1.2

Assumptions/Comments

1. 170mm Sand circle - Average existing texture
2. Add binder for coarse texture. Reduce the binder for smooth texture

3. 10% HCV in traffic spectrums above
4. Chip size based on mid-range per chip size
5. Prime coats - use Emulsion with low bitumen content typically 55%

7.2 ASPHALT SURFACING

The type of dense graded asphalt wearing course, where permitted for use, shall be selected in accordance with Table 19 unless otherwise specified. Other types of asphalt surfacing may be selected in accordance with NZTA M10 specification unless otherwise specified.

All asphalt mixes shall be designed in accordance with NZTA M10, NZTA M01-A and relevant standards for aggregate, mineral filler and binder referred in and shall meet the mix design requirements specified in the NZ and international standards. Mix design to be based on performance-based design criteria described in Guide to Pavement Technology Part 4B Asphalt.

Recycled asphalt pavement (RAP) up to 30% can be used for all asphalt concrete works. If an alternative asphalt surfacing is proposed for a granular pavement which is not described in Table 20, then laboratory tests for deformation and fatigue shall be undertaken. The results of these tests shall conclusively show that the alternative asphalt has equivalent or better resistance to deformation (Wheel Track Test) and fatigue (Repeated Flexural Bending Test) than the designated asphalt surfacing type.

Thin asphalt surfacing less than 40mm thick including open graded asphalt should not be considered as pavement layer in the structural design of the pavement.

7.3 CONCRETE SURFACING

Concrete pavement finished surface requires to meet the minimum texture and skid resistance requirements as detailed in the TDM.

All concrete should be laid in accordance with Standards NZS3109 and textured in accordance with NZS3114. In addition to these guidelines, further detail on the required texture for external finished surface must be specified as below.

- Non-slip surface with hard heavy broom finished surface U6 on across the traffic direction on public places including driveways, and pedestrian are and parking.
- Non-slip U6 hard broom finished surface across the traffic direction (at least 2 directions) or exposed aggregate finished surface intersections, and roundabouts,
- Bush Hammer U9 finished surface on public spaces including pedestrian crossing and path on slope > 5% gradient. We advised to refer AT TDM Footpath and Public Realms for minimum skid resistance requirements on finished surface and NZS Slip Resistance testing
- Exposed aggregate – aggregate exposed at time of placement U5E or F5E – public spaces especially approach to RSP tables where high skid resistance is required and pedestrian crossing where specified.

All concrete is screeded (U1) and mechanically floated (U2) prior to application of the specified texture. However, it is not acceptable to have finished surface U1, U2 and U3 for external concrete pavement.

8.0 CONSTRUCTION

8.1 SUBSOIL DRAINAGE

8.1.1 Construction

Piped subsoil drains shall be constructed in accordance with the TDM, NZTA F/2 specification and the details presented in the construction drawings. Subsoil drains shall be constructed with a preferred minimum grade of 1% and where this is not possible an absolute minimum grade of 0.5%.

Subsoil drains shall connect to catchpits above the soffit level of the outlet pipe at the outflow end only and shall be capped at the other end, which is also connected to the upstream catchpit as per TDM standards. Subsoil drainage shall be laid at an even grade without depressions that could not pond water and shall discharge into catchpits at spacings not exceeding 100m. Refer Section 2.6.

Where subsoils intercept more porous adjacent subgrade or trench backfill materials the subsoil trenches shall be locally wrapped with a class C geotextile cloth to prevent migration of the filter material. Transverse subsoil drains shall be provided where a change in pavement type results in a shallower pavement down gradient of a deeper pavement, where permeable central medians are provided or where localised undercutting of the subgrade does not extend to the longitudinal subsoil drain. Approved "Y" junctions shall be used to join transverse and longitudinal subsoil drains.

8.1.2 Testing

Quality control testing shall be carried out at the minimum frequency as follows:

- Geotextile – materials TNZ F7 Manufacturer's compliance certificate Check compliance certificate relating to each consignment.
- Subsoil drainage aggregate - materials TNZ F2 Crushing resistance 2 tests per material source and particle size distribution 1 test per100m^3.
- Backfilling, compaction to be done by visual check of backfilling and compaction.
- Subsurface pipes construction as per TNZ F2 Pipe type, location, gradient, damage, outlet and dockets to confirm pipe class.
- Visual check on flow of sub-surface during subbase and basecourse pavement construction.

Results of the testing shall be submitted to the Engineer for review as soon as possible following testing. Operations shall be interrupted and/or diverted as necessary to permit the tests to be carried out with complete safety and accuracy of test results.

8.2 GEOTEXTILES

For all subgrades exposed during construction, with CBR values less than 3.5%, a Class C geotextile should be placed to prevent contamination of the overlaying subbase course materials. A geotextile should also be considered on higher subgrade strengths depending on the potential for contamination, and reduction in strength of the subgrade during winter season construction before overlaying with the subbase course material.

8.3 GEOGRID

Where geogrid is required for subgrade improvement with a design CBR less than 3.5% then NZTA or AT approved triaxial geogrid should be used. The physical and mechanical parameters of the geogrid shall be appropriate for the overlay material usually less than 50mm.

8.4 DEFLECTION REQUIREMENTS

For construction testing the 95th percentile Benkelman Beam results, at the top of the basecourse, prior to surfacing, shall be less than the deflection listed in Table 25. In addition, the maximum deflection allowable shall be not more than 10% above the deflections provided in the table. If FWD is used for deflection testing, then the conversion to Benkelman Beam deflections should follow as per Section 6.

Auckland Transport Asset Acceptance and Asset Management requires to carry out falling weight deflection testing at 20m staggered lane on finished surface on flexible pavements prior to handing over the asset to the Auckland Council. A full FWD report with back analysed remaining life to be provided to the Auckland Transport during final inspection prior to road carriageway pavement asset is transferred.

Table 27: Deflection on Finished Surface on Flexible Pavement

Road Hierarchy	One Network Road Classification	Deflection 90th percentile (mm)	Maximum Deflection (mm)
Arterial & Primary	Arterial	0.70	0.80
Collector (Business)	Primary Collector		
Local (Business)	Primary Collector	0.80	0.90
Collector / Local (with buses*)	Primary Collector		
Collector without buses	Secondary Collector	1.00	1.10
Local roads and carparks	Access	1.25	1.40

8.5 SKID RESISTANCE

8.5.1 Macrotexture

At the end of the defect liability period, the macrotexture of the finished road surface or mean profile depth (MPD) must comply NZTAT10 Specification Table 3 Macrotexture Requirements provided in the Table 28 as follows.

Table 28: Finished Road Surface Minimum Macrotexture – mean profile depth (MPD)

Permanent Speed Limits	Chip sealed surface	Asphalt Concrete surface
50kph and less	1mm	0.5mm
Less than or equal to 70kph but more than 50kph	1mm	0.7mm
Greater than 70kph	1mm	0.9mm

Any paving that does not meet the performance criteria must be rectified at no additional expense to Auckland Transport. Flushed asphalt concrete or chip sealed surfaces are not acceptable and must be replaced. Water cutting is not an acceptable solution for treating flushed surface as this is only a temporary rectification providing much shorter design life with compliant micro texture.

8.5.2 Micro Texture

The finished surface must comply with the surface skid resistance requirements as set out in NZTA T10 Specification for State Highway Skid Resistance Management. Aggregate selection should be based on the aggregate performance method. Information on the performance of aggregates can be found on the website https://www.aggregates.stantec.online .

The stone source must be advised by the Auckland Transport representative for approval before use. If stone providing on road performance for the Site Category cannot be found, then special approval for an alternative based on the aggregate performance method should be sought from Auckland Transport. Aggregate for each asphalt wearing course Job Mix Formula must meet the NZTA T10 requirements.

Concrete finishes need to be treated with caution as these surfaces can be texturised by an exposed aggregate finish, grooving (as in airfield pavements), diamond grinding, or applying calcined bauxite micro-surfacing. However, such treatments may not be required on roads with low-speed environments unless there is specific reason for such application to be confirmed by the Auckland Transport Asset Owners and/or Auckland Council.

8.5.3 High Friction Surfacing

High Friction Surfacing (HSF) is used on roads to increase the skid resistance of the road surface, especially to assist drivers stopping the vehicles to reduce the risk of accidents at high speeds. This HSF surface is expected to contribute to a safer driving environment on busy roads.

HFS treatments are pavement treatments that dramatically and immediately reduce crashes, injuries, and fatalities associated with friction demand issues, such as:

- A reduction in pavement friction during wet conditions, and/or
- A high friction demands due to vehicle speed and/or roadway geometrics.

HFS involves the application of very high-quality aggregate with high PSV to the pavement using a polymer binder or epoxy resin to restore and/or maintain pavement friction on potentially high crash areas. The higher pavement friction helps motorists maintain better vehicle control in both dry and wet driving conditions.

The treatments can be applied using either hand or mechanical methods. The speed of a fully mechanised HFS installation is like other paving surface operations. Various types of HFS systems are available based on the aggregate types, binder systems, and application methods – generally proprietary specification and standards and can vary depending on how the product is adopted for treatment. HFS products are available from numerous manufacturers and contractor suppliers.

8.5.3.1 Application methods and Specifications

High friction surfacing such as calcined bauxite shall be placed where identified by Auckland Transport Traffic Safety Engineers as hazard or accident spot or on the sharp bend with super-elevation or other areas such as approaches to crossings. This should be laid on a dense graded asphalt, preferably a NZTA M10 DG10, AC10 or AC14 surface. The materials and specification on high skid resistance to be as per NZTA P-25 Pilot Specification.

High friction surface coating can last for eight to ten years depending on the surface and the skill of those laying it on the road. However, the life span of HFS is affected by several factors especially materials – epoxy binder used and construction methodology, quality of substrate preparation, the volume of traffic etc. For AT projects, it is advised to specify in HSF surfacing contract with a minimum design life of 8 years with a defect liability period of 2 years.

Reference can be made to Austroads Publication no: ATS-3466-23 High Friction Surface Treatment and NZTA P25 Specifications for High Skid Resistance Surfacing.

Where warranted for improvement to skid resistance, SMA10 surfacing can be applied on existing asphalt surface following the surfacing design and approval from AT design team.

8.5.4 Colour Surfacing

Colour surfacing is generally provided as an added feature to emphasise direction and warning to the pedestrians and road users. It can help to direct or warn road users in permanent treatments, or to visually enhance streets and public places. This has been in the road network for both road traffic and active transport as follows:

- Coloured surfacing at busy driveways to warn drivers to give way to pedestrians when crossing a footpath or shared path.
- Coloured surfacing at bus stops to allocate space for different users (e.g. between pedestrians and cyclists) including those with a wide range of disabilities
- Red surfacing on the approaches to a zebra crossing for slowing the vehicle speed

Following NZTA P33 Specification for Coloured Pavement Surfacing and guidance notes published by NZTA (refer to https://nzta.govt.nz/resources/coloured-surfacing-principles/) set out the principles for the use of colour, including type of colour applies and detailed design advice and standards. It also includes coloured surfacing specifications, materials, maintenance, and installation.

- New Zealand Pedestrian Network Guidance (PNG)
- Cycling Network Guidance

A performance specification NZTA P30 is available in which the expected life of colour surfacing provided by the supplier shall be minimum design life of 5 years.

Note that any coloured markings within the roadway or directing traffic need to comply with the Traffic Control Devices (TCD) Rule. For more detailed guidance on coloured surfacing in these situations refer to the Coloured surfacing principles best practice guidance note, where you can find information on the legislative context, application for different purposes, colour specifications and material types, and installation and maintenance.

8.6 ASPHALT

Asphalt concrete paving including compaction and quality control shall meet the requirements of the NZTA M10 specification. The asphalt bitumen binder shall meet the NZTA M01-A specification.

Only Auckland Transport approved asphalt mixes including stone mastic asphalt concrete are permitted for use. All submitted mix designs by asphalt suppliers are reviewed and validated annually.

Open Graded Porous Asphalt (OGPA) mixes are not currently within the AT approved list of mixes. For movement category M3 carriageways, project specific approval is required from the Auckland Transport Asset Management team for their use including review and acceptance of the mix designs.

The asphalt layers must be produced, paved, and tested according to the requirements of NZTA M10 and minimum design life for all thin asphalt surfacing shall provide a design life of minimum 12.5 years for major road including any bus routes under movement category M2/ M3 classification and 15 years for local residential streets under movement category M1/M2 classification.

8.6.1 Surface Ride

The new and rehabilitated pavement must have an average dynamic roughness, when measured over a length of 100m, of less than 60 NAASRA counts/km for any three consecutive results and no individual value greater than 70 NAASRA within the extent of the re-surfacing area unless it can be clearly attributable to a permanent feature such as a bridge joint.

8.6.2 Surface Ride for Resurfacing Sites

The pre-resurfacing site roughness measure must be obtained from Auckland Transport RAMM database – high speed roughness count. Where these measures do not exist, testing must be performed. The average roughness count must be used to benchmark the resurfacing works, as described below.

The roughness measurements of all new surfacing must be carried out on completion of the surfacing. All results must be submitted to the Auckland Transport representative within 2 working days.

The new surface when measured over a length of 100m must achieve an average NAASRA roughness less than the value calculated using the formula below. No two consecutive counts must exceed 70 and no individual count greater than 80 within the extent of the resurfacing are permitted unless this can be clearly attributable to a permanent feature such as a bridge joint.

NAASRA Count Criteria = 0.7D + 5 (D = average NAASRA roughness measure determined before the commencement of asphalt resurfacing.)

Where the roughness improvement criteria are not satisfied, remedial works must be undertaken to bring the roughness to the acceptable limit at no additional cost to Auckland Transport.

8.6.3 Surface Irregularities

The new pavement must be free from depressions or areas that pond water, any abrupt surface level, including service covers and irregularities exceeding 6mm when measured with a 5m straight edge.

All service covers must be raised during new surfacing or resurfacing operations to be flush with the adjacent finished pavement surface level.

8.6.4 Density

The density requirements for the compacted mat are as defined in the NZTA M10 specification or as stated in the specific contract requirements.

8.6.5 Flushing, Shoving, Segregation and other Defects

The asphalt surfacing must not exhibit any signs of flushing, shoving or segregation following completion of the works and at completion of the defect liability period. Water cutting is not an acceptable remedy for flushed surfaces.

8.6.6 Texture

At the end of the defect liability period, the surface texture of the finished road surface must comply with the minimum texture depth provided in Table 28.

Any paving that does not meet the performance criteria must be rectified at no additional expense to Auckland Transport. Flushed SMA type surfaces are not acceptable and must be replaced. Water cutting is not an acceptable solution for treating flushed SMA type mixes.

8.6.7 Skid Resistance

The finished surface must comply with the surface resistance requirements as set out in NZTA T10.

8.7 CHIPSEALING

Chip sealing here refers to membrane sealing either single or double coat and resurfacing.

8.7.1 Chip Retention

The sealed area must have a uniform retained layer of chip. The requirement for acceptance must be such that the area covered by chip in close shoulder to shoulder contact must be not less than 98% of the total area considered. The minimum area to be considered must be 300mm x 300mm.

8.7.2 Surface Texture

When measured in accordance with the procedures specified in NZTA T3, the finished chip sealed surface can be expected to perform acceptably for a period of not less than the design life.

8.7.3 Remedial Work

Any remedial work undertaken on the resealed surface must have an equal standard of safety, durability, waterproofing, roughness and texture within + 15% of the sand circle of the surrounding surface to that of an undamaged resealed surface and must be virtually indistinguishable from the adjacent surface.

8.7.4 Construction Quality

No obvious defects resulting from poorly constructed longitudinal or transverse joints, blocked or inappropriate spray nozzles, or incorrect chip spreading must be visible. The quality of resealing must meet the minimum requirements of the Inspection and Test Plan in Appendix A Table 32 & Table 33.

8.7.5 Safety

Procedures for heating, blending, spraying and transferring binder materials must comply with "Best Practice Guideline: Safe Handling of Bituminous Materials Used for Roading (Civil Contractors New Zealand).

Considering the Climate Change and ongoing improvements with better performing emulsion binders, cutback binders will not be accepted in after 2024. Bitumen emulsions shall be used for network surfacing unless otherwise approved by Auckland Transport.

8.7.6 Timeliness

Within a period of 48 hours from the time of completion of sealing, the road must be swept of surplus chip and have pavement marking reinstated as existing. This period may be extended for roads carrying particularly low traffic volumes that would benefit from or require a longer 'bedding in' time prior to sweeping.

Sealing records including daily site sheets and forms shall be submitted by the due date. Quality Assurance records and test results shall be submitted by the due date.

Acceptance inspections must be performed on the initial completion of the work, and 12 months after completion of the work.

8.7.7 Membrane Seal

Membrane sealing intended only for overlaying with asphalt concrete requires less chip spread relative to the permanent sealed pavement surface and therefore over chips to be swept and removed prior to the paving with asphalt concrete.

8.8 SLURRY SURFACING

Slurry seals have been generally applied as a thin wearing course, typically on low volume roads such as residential streets as either preventative maintenance on existing sound pavements, or as corrective maintenance. Slurry seals typically utilise smaller sized aggregate for areas such as car parking lots, school yards and bicycle paths. Slurry seals may also be suitable on roads to restore skid resistance and to serve as a preventative maintenance treatment. Slurry sealing use finer aggregates because of the reduced chance of being plucked out of the surface.

Whilst slurry seals can be used to seal minor surface cracks, they will not stop reflective cracking. It is preferable that all but very minor surface cracks be sealed with an appropriate crack filler/sealant prior to application of the slurry seal. The surface texture depth of a slurry seal may not be appropriate for high-speed roads.

8.9 MICROSURFACING

Microsurfacing treatment is a bituminous thin surfacing that contains polymer modified emulsion binder, which is capable of being spread in layers with variable thickness for rut-filling and correction courses, and for wearing course applications requiring good surface texture. This treatment can be primarily used as resurfacing existing asphalt concrete as a temporary sealing using polymer modified binder with fine aggregates for maintenance intervention until permanent surfacing is taken place.

Microsurfacing has the advantage over slurry seals with improved binder characteristics through the incorporation of polymer. This also allows the use of larger nominal sized aggregates and enables its application in higher traffic areas up to 2000 AADT and in layers up to 20 mm thick. Micro-surfacing can address several pavement maintenance requirements that cannot be readily met by sprayed seals or asphalt, e.g. minor shape correction while matching into existing levels.

For detailed understanding with design and construction, it is advised to refer Austroads Guidelines and Specification for Microsurfacing. Recently, AT has carried out a few trials in the local residential streets and will be monitoring the performance of the surfacing and texture as well as general perception of the communities.

Emerging technologies with bitumen production and using additives such as fibres may improve flexibility or strength of material in the applied microsurfacing and are generally proprietary. However, a mix design for microsurfacing is required prior to commencing works, and this should form a hold point for works, i.e. work is not to commence until the mix design is submitted. Testing of mix designs should be carried out by a NATA accredited laboratory.

8.10 SURFACE FINISH

8.10.1 Surface Irregularities

The new pavement must be free from depressions or areas that pond water. Any abrupt change in surface level, including service covers and irregularities shall not exceed 6mm when measured with a 5m straight edge.

All service covers must be raised during new surfacing or resurfacing operations, if required, to be flush with the adjacent finished pavement surface level.

8.10.2 Flushing, Shoving, Ravelling, Segregation, and other Defects

The asphalt surfacing must not exhibit any signs of flushing, shoving, ravelling or segregation following completion of the works and at completion of the defect liability period. Water cutting is not an acceptable remedy for flushed surfaces.

9.0 CONSTRUCTION QUALITY ASSURANCE

Construction quality assurance shall follow the principals detailed in NZTA Z1 (2021) *Minimum Standard for Quality Management Plans* and NZTA Z8 *Minimum Standard for Inspection, Sampling and Testing*.

Quality assurance shall be increased for RASF category P3 and M3 environments especially for primary collector and arterials roads. This is to ensure that the construction monitoring is focused on the appropriate factors and ensuring a quality pavement outcome. The material and construction requirements are detailed in the inspection and test plan template so contractors will be able to establish testing quantities prior to construction.

As an example, Table references the applicable specifications for various pavement layers.

9.1 TOLERANCE

The following guidance is included to assist pavement designers in preparing these Schedules.

9.1.1 Design Levels, Layer Design Thickness and Critical Layer

To allow for variations in the constructed layer thicknesses within the construction level tolerances, 10 mm shall be added to the pavement layer which governs the overall allowable loading (designated the critical layer). Layer thickness shall conform to the specification limits, construction practicalities and performance requirements. The following pavement design layer thickness constraints and preference shall apply:

- Thin asphalt on granular pavements – for pavements designed in accordance with Austroads AGPT-2 or this TDM Guide, the minimum design thicknesses of 40mm for DG10 or AC10 and 55mm for AC14 shall apply. AC14 50mm can be paved over the existing asphalt basecourse.
- Unbound granular AT AP40 or NZTA M4 - AP40: 125mm (minimum thickness) - 200mm (each layer)
- Unbound granular AT AP65 or NZTA M3 – GAP65: 160mm (minimum thickness) - 250mm (each layer)
- Plant mixed cement stabilised AP40 – single layer construction 125 – 200mm.

9.1.2 Level Tolerances

Pavement construction layer thickness and finished surface geometry to be verified using level survey at subgrade level and on each completed pavement layer and the finished surface level. Design levels are specified on pavement schedules, with a tolerance applied to each level. As-constructed levels that fall within the design level plus tolerances are conforming, as-constructed levels that fall outside of design levels plus tolerances are non-conforming. Pavement level layer tolerances shall be as follows:

- Surfacing (wearing course) – thickness -0 and +10mm and level tolerances -0, +5mm.
- basecourse – thickness +/- 10mm and level tolerance to -5 and +5mm.
- subbase course – thickness +/-10mm and level tolerance -10 and +10mm.
- Subgrade preparation – level tolerance -10 and +10mm.

The designer must consider the overall pavement structure, and which layer is the critical layer, as well as constructability, quality management systems and other factors, for each specific configuration to ensure these are suitable. Tighter tolerances may apply to the surface level and the bottom level of the critical pavement layer, which controls the total pavement thickness.

Table 29: Specification References for Various Pavement Layers

Pavement layer	Material Specification	Construction Specification
Basecourse AP40	NZTA M4 or TDM (AT AP40)	NZTA B2
Subbase AP65	TDM (AT AP65)	NZTA B2
Dense graded asphalt	NZTA M10 NZTA M01-A	NZTA M10
Stone mastic asphalt	NZTA M27 NZTA M01-A	NZTA M27
In situ stabilised (modified) basecourse or subbase	New Zealand Guides and project specification	NZTA B5
Ex situ stabilised (modified) basecourse or subbase	New Zealand Guides and project specification	NZTA B7
Stabilised subbase	AT TDM	NZTA B6
Subgrade	-	NZTA F1 and or NZTA B9
Stabilised Subgrade	AT TDM	NZTA B9
Subsoils	AT TDM	TDM requirements with NZTA F2 filter material backfill to trenches and geotextile sock on pipe. Other types of drainage material may be acceptable for rural roads subject to the site condition.

An indicative inspection and test plan for pavement construction and material testing is included in Appendix A. The remaining specification requirements are detailed in the Inspection and Test Plan template.

9.2 SUSTAINABILITY

Consideration of existing and potential means of promoting sustainability in pavement design, materials, pavement and drainage maintenance activities and construction should be given by designers and others involved in determining what materials and processes go into these activities.

The pavement design, materials, pavement and drainage maintenance activities and construction approaches for all maintenance and renewals works should align with the following sustainability outcome:

- Target zero waste through waste minimisation and/or recycling/re-use maximization.
- Energy savings using lower temperature mixes.
- Reduction in contribution to greenhouse gases production.
- Contribution to whole of life cost reductions for maintenance of road transport assets.
- Usage of biofuels and other similar fuel derivatives.

Given the numerous existing and potential sustainable inputs into road pavements, this might include consideration of some of the following:

- The use of recycled materials in road construction, such as asphalt, in the production of recycled asphalt pavement (RAP) mixes has already been permitted by existing NZTA specifications. Similarly, aggregates, with the use of recycled crushed concrete, the re-processing of waste aggregates and the use of industrial by-products (such as slag in asphalt).

- The use of various materials in construction, such as subgrade undercut situations where a variety of suitable materials could be made available including use of glass sand and millings products.
- The use of stabilisation/modification of pavement aggregate is promoted within the provisions of NZTA specifications such as NZTA M4 to make locally available aggregates suitable for use as basecourse material. The use of slightly lesser grades of pavement aggregate with some modification may be suitable for use in some applications such as local residential streets with low volume of traffic where the use of premium aggregate is unnecessary. Provisions are made in this document to allow this.
- The stabilisation of existing road materials, including in-situ stabilisation, incorporates the current use of some recycled materials or by-products as additives.

10.0 SHALLOW SERVICE PROTECTION

10.1 SHALLOW SERVICE

It has observed that due to infilled dense housing construction in the established urban area, new utility connections are required to connect to the existing, which requires to encroach into the road carriageways.

TDM standards requires all chambers including manholes to be outside the road carriageways. SW Code of Practice and National Code of Practice for Utility Operators by NZ Utility Advisory Group (NZUAG) 2010 and provide guidelines for reinstatement of the trenching and chambers in the road carriageways.

All utility services shall be installed at least 1m below the finished road surface level. Cover to the major services such as high voltage cables and water main / gas main to be preferably 1.4m or not less than 1.2m from the road surface as per the utility owners' specification and standards.

On berms and concrete footpath, the cover depth can be reduced to minimum 600mm provided there is unlikely to have any heavy vehicle traffic over the buried berm and footpath area.

Protection measures are to be provided for any shallow services in the carriageway by means of concrete capping to the service as well as protection of any differential settlement and any pavement failure due to consolidation of the backfill materials.

A practice note will be published on the design requirements for reinstatement of road pavement for all shallow services including service chambers and manholes in the carriageway. Departure from the Standards (DfS) to be applied for with mitigation measures for any services requiring to installed in the carriageway with less than 1m cover to the finished road surface and any chambers with lid.

For footpath/ cycle path and berms, such services can be as shallow as 0.6m provided there will be unlikely to have any heavy vehicles movements in such areas.

10.2 SERVICE CHAMBERS AND LIDS

It is not desirable to have any type of service chamber or manhole in the traffic lane or road carriageway especially within the wheel path due to its difficulties with access and any maintenance requiring temporary traffic management and disruption to the road traffic. All chamber lid to be classified for minimum HN-HO-72 loading or Class D and to have lockable and unremovable hinged heavy-duty cast-iron lid.

All service chambers including manholes are to be installed outside the road carriageway traffic area especially on new / greenfield construction. If this is not possible then service chamber may be installed away from the main traffic lane or outside the wheel paths or preferably at the centre line of the carriageway such as in traffic or median island.

Special treatment to the pavement and surface using structural asphalt concrete including surfacing at least 1m around the chamber cover lid to prevent pavement deformation / settlement due to consolidation of the backfilled materials around all service chamber/ manhole and lid to be considered in the reinstatement design as mitigation measure. In addition, subject to applying for obtaining approval of DfS, which is not automatically granted to installation of such service chambers and manholes within the road carriageway. A reinstatement details around the chamber / manholes showing backfilling and pavement details to be submitted along with departure from the standards (DfS) application for the AT review approval.

10.3 UTIITY DUCTS AND PIPES

All buried pipes are subject to loading from the weight of soil overburden. When pipelines cross railroads, roads, parking lots, or construction sites, the pipes also experience live surface loading from vehicles on the ground, including heavy construction equipment in some scenarios. The surface loading results in through-wall pipe bending, generating hoop and longitudinal stress.

Auckland Council's Stormwater Code of Practice states that all utility ducts and stormwater pipes shall be located where practicable and viable within road reserves or other public land but not in the road carriageway in the traffic lanes. It has been recommended to allow at least 600mm clearance to the road surface to enable concrete capping as protection to the utility services pipes and construction and traffic loads. Bedding details are also to be considered to ensure there are no joint displacements due to impact loading from the heavy traffic loads.

Depth of cover less than 900mm is generally not recommended under major roads due to the possibility of high dynamic loading, which could result in damage to the pavements and/or the pipes. It is advised to carry out design calculation for trench reinstatement and classification and materials of pipes to be used allowing a dynamic load impact factor of 1.5 and at least 12-tonnes single axle load wheel load including any heavy machines and plant likely to be used for pavement construction. It has been observed the CCTV surveys that many new concrete pipes installed at shallow depths despite their high-grade classifications were cracked longitudinally over the periods due to various loading including during the construction. Therefore, consideration of using appropriate and stronger pipe material such as high-grade corrugated polyethylene or HDPE pipes could be considered when such pipes are in the road carriageway and are continuously subject to heavy vehicle loading.

Standard concrete capping with 150mm minimum 20MPa concrete with a layer of Grade 500 661 mesh over the pipes is recommend when the cover depths are less than 900mm and pipe diameter is more than 200mm. A typical detailed standard drawing for such trench and pavement reinstatement are being prepared by Auckland Transport and will be made available in the TDM in the future.

11.0 PAVEMENT MATERIALS

11.1 AGGREGATE SUPPLIES

The Auckland Transport or Auckland Council representative may require sampling aggregate material both at the source and at the Site at any time during the construction of a pavement layer and to have tests carried out by a IANZ independent accredited laboratory to verify compliance with the requirements specified.

Failure of any aggregate material to meet the specified requirements will result in the Auckland Transport representative ordering the removal of such material from Site. It must be removed from site and be replaced with material that does comply all at no cost to Auckland Transport.

All aggregates shall comply to TDM 800 Series: Aggregate Specification (Draft – 2024).

11.2 PAVEMENT SPECIFICATIONS

Unless specified in the Project Specification, this specification shall be read in conjunction with the following specifications:

- Contract Drawings, Basis of Payment and Schedule of Prices
- NZS 3122. Specification for Portland and blended cements (General and special purpose)
- NZS 4402 - Methods of testing soils for civil engineering purposes
- NZS 4407 - Methods of sampling and testing road aggregates
- NZTA M01A - Specification for performance graded binders
- NZTA M10 - Specification for dense graded asphaltic concrete
- NZTA M27 - Specification for Stone mastic asphalt
- NZTA P25 - Pilot specification for calcined bauxite to be used for High Friction Surfacing
- Transit T1 - Benkelman beam deflection measurements
- NZTA T10 – Specification for state highway skid resistance management
- NZTA T15 – Specification for repeated load triaxial (RLT) testing for pavement materials
- NZTA T19 – Procedures for design and indirect tensile strength testing of modified and bound pavement materials
- Transit F1 - Specification for earthworks construction
- NZTA M4 – Specification for basecourse aggregate
- NZTA M3 - Specification for subbase course aggregate
- Transit B2 - Specification for construction of unbound granular pavement layers
- NZTA B5 - Specification for in situ stabilisation of modified pavement layers
- NZTA B6 - Specification for in situ stabilisation of bound sub-base layers
- NZTA B7 - Specification for the manufacture and construction of plant mixed modified pavement layers
- NZTA B8 - Specification for the manufacture and construction of plant mixed bound sub-base pavement layers
- NZTA B9 – Pilot specification for in situ subgrade stabilisation

The current specification at the time of design shall be applicable during design and construction.

This specification shall include the following:

- Rehabilitation of an existing pavement or construction of a pavement above a formed/ existing subgrade level.
- Supply of all materials, tools, plant, labour and supervision for the completion of the whole works in accordance with the contract specification.

The sequence of pavement construction shall be managed to ensure that there is no contamination of the various pavement layers during the construction of the current layer.

11.3 SUBGRADE

In the context of pavement design, the subgrade is defined as the layer of material 1 m deep below the underside of the finished level. Therefore, the subgrade shall include any modified subgrade layers and/or subgrade improvement layers within that depth.

11.3.1 Subgrade Preparation

Cut to waste material is to be removed from site to an approved dumpsite found and managed by the Contractor.

The subgrade shall be trimmed to line and level to achieve an even grade between level control points. Once the surface compaction is complete, the finished levels of the excavated sub-grade surface shall be within a tolerance of +0 mm - 20 mm of the design levels as determined in accordance with the design profiles shown on the drawings.

In addition, the finished surface gradients and crossfalls shall not vary locally from the design gradients and crossfalls to the extent that water is able to pond at any point.

The subgrade is to be dried back as required and compacted to achieve maximum dry density (MDD) once compacted. The subgrade shall then be proof rolled using a static, steel wheel roller. Any soft areas observed during rolling of the subgrade shall be referred to the Engineer.

Care should be taken not to over-wet any clayey soils encountered as this can lead to problems associated with trafficability and workability. Clayey soils should not be over-compacted or placed too dry compared with the OMC, as this can lead to future swelling and softening with changes to moisture content.

11.3.2 Subgrade Testing

Scala penetrometer tests are to be conducted on all areas of the subgrade to determine the consistency of the CBR strength. Scala penetrometer tests shall be carried to a depth of minimum 1.5m below the design bottom of the pavement. A hole for the Scala penetrometer should be pre-bored if the depth to be tested is greater than 1.5m from existing ground level as skin friction can compromise measured accuracy.

The Engineer may additionally request the Contractor to carry out laboratory soaked CBR tests with associated swell and shrinkage testing to assess the actual swell/shrinkage potential of the subgrade material.

The Engineer shall review the results of the Scala testing and proof rolling and, where the target CBR is not achieved, determine the extent of any further undercuts required or alternatively recommend a modification of the pavement layer thicknesses.

Depressions formed by the removal of existing structures, pavements, underground elements etc. will require all disturbed, weakened soils to be excavated and removed, then backfilled with compacted select filling.

The Contractor shall not commence the construction of subsequent layers until the Engineer's acceptance and approval of the Scala test results and underlying surface is obtained.

Unless stabilisation improvements are achieved a Class C geotextile and geogrid combination should be used between the subgrade and subbase interface where the subgrade CBR strength is 3.5% or lower.

All sampling and testing shall be performed by an IANZ accredited laboratory.

11.3.2.1 Removal of existing granular material to stockpile

Where applicable, the existing granular pavement material in the areas of the development shall be cut to stockpile for reuse as subbase or backfill material. The suitability of the material shall be confirmed as specified in sections below. The granular material should be excavated ensuring no contamination of subgrade or topsoil and stored in an area free from contamination from other materials or water. The Contractor shall not commence stockpiling of material until the Engineer's acceptance and approval on the quality of the material is obtained.

The surface of the stockpile area should be pre-prepared to ensure that the recycled pavement materials do not become contaminated from storing on top of topsoil. The Contractor shall stockpile only in areas approved by the Engineer.

Any contaminated aggregates used for haulage road should be fully removed from the carriageway for new pavement construction and replaced with certified aggregate products as per the approved design.

11.3.2.2 Contaminated Soils

If the site is potentially contaminated, then the site should be tested to confirm or eliminate the risk of contamination.

If contamination is found the Engineer shall be notified immediately, and the material shall be removed (not stockpiled) immediately after excavation and disposed of offsite at an approved fill site in accordance with the Ministry for the Environment (MfE) Contaminated Land Guidelines.

The Contractor shall maintain full and accurate details of each load as evidence of proper disposal and provide all disposal records to the Engineer to allow the Resource Consent to be closed out.

If any contamination testing is required, it shall be conducted by a contaminated land professional in accordance with the MfE Contaminated Land Guidelines.

11.3.3 Subgrade Improvement Layer

Where insufficient compaction or design subgrade strength has not been achieved, undercutting of the subgrade and backfill with granular material is expected. As above, The Contractor shall get written approval from the Engineer prior to commencement of these works.

The subgrade improvement layer may be constructed by modification or stabilisation of the subgrade using either chemicals or gravel / rock backfilling materials, imported granular material or Woodhill black sand, or other imported approved material. Where imported material is to be constructed on subgrades of CBR 3.5% or less a Class C geotextile shall be used under the imported material. Where sand is used for subgrade improvement care shall be taken in design to ensure it is confined and cannot be mobilised by groundwater movement.

Reactivity testing in the IANZ accredited laboratory is suggested to determine the type and quantity of the additives to be used for the targeted subgrade strength.

Granular fill or black sand for the backfill of the undercut subgrade to be confirmed by the Engineer prior to construction of the subgrade improvement layer.

Necessities for modification / stabilisation and the necessary additive for the process is to be confirmed by the Engineer prior to construction of the subgrade improvement layer.

Unless Auckland Transport approval has been obtained for the design and use of subgrade improvement strategies, the Engineer may be required to obtain approval prior to construction. Contingency and risk planning is critical in developing the scheme and detailed designs.

11.3.3.1 Granular Fill (Imported hardfill)

The Contractor shall select, supply, place, condition, compact and maintain granular fill to meet the performance criteria specified below.

- The fill material shall have a maximum aggregate size of 100mm.
- Well graded (within 0.4 and 0.7 slope on a log/log grading envelope), must be spread and compacted in layers of uniform thickness, no greater than 200mm.
- Soaked 4-day CBR of not less than 50%.
- The compaction characteristics and properties (such as the sand equivalent, crushing resistance, weathering) of the granular fill shall be provided by the Contractor to the Engineer prior to delivery of any material to site.
- The material shall not contain organic matter or deleterious material.
- Any other suitable selected subgrade improvement layer or engineered fill to be confirmed by the Engineer and approved by Auckland Transport.

11.3.3.2 Modified Subgrade Improvement

The Contractor shall select, supply, place, modify / stabilise, compact, and maintain subgrade improvement layer to meet the performance criteria specified below.

- Stabilisation additives may include but not limited to cement and lime.
- Soaked CBR of not less than 10%. Test should involve soaked CBR - 3 days bench cure then 4 days soaked. Test should have a 4kg surcharge and test should follow NZS4402 1986 Test 6.1.1 CBR and 4.1.1 standard compaction.
- The material shall be sampled directly after stabilisation and the samples compacted by the laboratory within 2 hours of field mixing.
- Reactivity test to be carry out for CBR following 4 days of curing both soaked and unsoaked conditions.
- The material shall not contain organic matter or deleterious material.

11.3.3.3 Black Sand

A large part of the Takanini area and part of Papakura is underlain with peat and the focus of this report is best practice for the design and construction of roads in this area. Refer the Indicative Peat Soil area in Soils Map in Appendix D.

Subgrade materials in this area primarily comprised of peat soil with very weak subgrade strength – CBR < 1% and carriageway pavement including other infrastructure cannot be built without improvement to the existing subgrade.

Historically, pavement on peat subgrade has been constructed with lower part of the pavement comprising a layer of graded sand (Woodhill / black sand) – typically 300mm to 500mm placed directly on the subgrade. The depth of these layers depends on the stiffness of the underlying peat and volume of heavy vehicle traffic on the pavement.

The benefit of using sand is that it can be compacted with much portable hand-held equipment such as a plate compactor, which reduces the risk of damaging the subgrade during construction. The sand also performs well when it is saturated and is therefore suitable to areas with high ground water levels. As the sand is wetted sufficiently during construction, it can achieve a moderate to high stiffness improvement relative to the soft subgrade with inferred CBR more than 5 when tested using DCP or Clegg Hammer. Pavement design can be carryout out as per the standard pavement design procedures using mechanistic modelling using CIRCY.

During construction, it is important for this layer to be confined with geotextile to prevent erosion or loss of sand over time to ensure that the sand as subgrade improvement layer performs at least for the design life of the pavement. It will be necessary to use a geotextile next to subsoil drains, over trenches and anywhere else where the sand is likely to be eroded or left out due to sub-surface water or the traffic loading.

All remaining pavement layers can be constructed as per the standard granular pavement construction with care not to damage the compacted subgrade improvement layer with sand. Generally, for road with bus services, it is recommended lay CTB treated subbase course minimum 250mm using AP40 aggregates instead of in-situ stabilisation and then overlaying with basecourse layer. Deflection on finished subbase course to be checked against minimum acceptance criteria as provided in Table 21.

As a design guide, depth of the subgrade improvement layer using black sand should be as follows:

- CBR <3% & >/= 2% - 300mm minimum Class C geotextile encased
- CBR >/=1% & < 2% - 500mm minimum Class C geotextile encased and geogrid at bottom of subbase course
- CBR < 1% - 700mm minimum Class C geotextile encased and then geogrid at bottom of subbase course.

11.4 GRANULAR PAVEMENT LAYERS

Granular pavement layers above the subgrade layer are generally composed of an AP65 (subbase course) and AP40 (basecourse) aggregates and in accordance with the Auckland Transport 0800 Aggregate Specification.

11.4.1 Subbase Course- Recycled/Imported

The granular subbase course materials shall be constructed according to the NZTA B2 specification.

11.4.2 Basecourse- Recycled/Imported

Basecourse aggregate for all RASF movement category M3 and M2 roads must comply with the requirements of NZTA M4 Class 2 unless instructed otherwise by the Auckland Transport representative. Basecourse aggregate for all roads movement category M1 roads must comply with the following requirements unless instructed otherwise by the Auckland Transport representative.

- Basecourse construction shall be undertaken as per design drawings in maximum 200mm deep lifts.
- The granular basecourse materials shall be constructed according to the NZTA B2 and NZTA M4 AP 40 Class 3 specification.

Repeated Load Triaxial (RLT) testing may be required by Auckland Transport to validate the designer's assumptions during the design phase or aggregate performance during the construction phase when an unbound or modified basecourse is considered as having a high risk of rutting. High risk scenarios may include, but are not limited to the following:

- Where the DESA is at or near the boundary between being a low and medium risk treatment
- Where the initial development or construction HCV loading account for more than 30% of the life of the pavement.
- If marginal or unproven recycled aggregate is used
- If there is a higher risks of moisture ingress (e.g., widening construction, or near waterways, flood plains or flood prone zones).

11.5 MODIFIED GRANULAR PAVEMENT LAYERS

This section sets out requirements for the modification of granular layers including cement modification or foamed bitumen stabilisation.

- The granular pavement shall be constructed according to the NZTA B5 specification.
- Bitumen must comply with NZTA M01-A specifications.
- The aggregate shall be sampled and tested prior to commencement of delivery. The rate of sampling shall be as stated in the minimum requirements shown in the ITP in the Appendices.
- All laboratory testing shall be completed by an IANZ endorsed laboratory under instruction from the Contractor and the results should be approved by the Engineer prior to construction.
- The foamed bitumen modified basecourse surface must have all Clegg Impact Values exceeding 40 prior to membrane seal or two coat chip seal.
- The foamed bitumen modified basecourse surface must meet deflection compliance prior to asphalt surfacing.

The GP cement content shall be determined using the ITS testing in accordance with NZTA T19, with indicative ITS test ranges for the modified basecourse aggregate as detailed in Table . Foamed bitumen mix design shall follow the methodology detailed in the NZ Guide to Pavement Evaluation and Treatment Design (NZTA 2018) and an Unconfined Compressive Strength (UCS) result is also required. ITS values outside of the ranges in Table can occur with the upper bound exceeded with minimal cement content, if this occurs it should be discussed with AT and a way forward agreed.

Table 30: Indicative ITS / Resilient Modulus Values

Pavement Layer Type	Dry ITS (kPa)	Soaked ITS (kPa)	Resilient Modulus (MPa)
Unbound Granular Basecourse	Not applicable	100 minimum	350
Cement / Lime Modified Basecourse	150 to 350	100 to 300	500
Foamed Bitumen Stabilised Basecourse	175 to 400	150 to 350	800

11.6 CEMENT BOUND SUBBASE

This section sets out requirements for the stabilisation either in-situ or plant mixed of subbase granular layers with cement stabilisation. Cement stabilisation is not permitted within the basecourse granular layer.

- The subbase materials shall be constructed according to the NZTA B6 or the NZTA B8 specifications.
- Plant mixed: The target minimum unconfined compressive strength (UCS) is 3 MPa after a 7-day cure. The mould size for the UCS testing must be 100mm in diameter and 200mm high. Compaction of the sample must be in accordance with NZS4402:1986 4.1.2 NZ Vibrating Hammer.
- Wet and Dry ITS testing shall follow the NZTA T19 specification. Compaction of the sample must be in accordance with NZS 4402 test 4.1.3 NZ Vibrating Hammer
- The aggregate shall be sampled and tested prior to commencement of delivery. The rate of sampling shall be as stated in the minimum requirements shown in the ITP.
- During stabilisation material should be sampled from behind the hoe and compacted within the time limits as detailed in the UCS technical note.
- All laboratory testing shall be completed by an IANZ endorsed laboratory under instruction from the Contractor and approved by the Engineer prior to construction.

The GP cement content shall be determined using the ITS testing in accordance with NZTA T19, with ITS test ranges for the stabilised basecourse aggregate as detailed in Table . Stabilisation should be achieved by a minimum of 3% cement.

Table 31: ITS Values for Stabilised Materials

Pavement Layer Type	Dry ITS (kPa)	Soaked ITS (kPa
Cement Bound Subbase	>500	>450

11.7 LEAN MIX CONCRETE SUBBASE

This section shall be used for the supply, delivery and construction of the Lean Mix Concrete.

All materials and workmanship shall comply with these Standards unless expressly noted otherwise:

NZS 3104:2003	Concrete Production
NZS 3109:1997	Concrete Construction
NZS 3111	Methods of test for water and aggregate for concrete
NZS 3112.1	Methods for test related to fresh concrete.
NZS 3112:2	Methods for test relating to the determination of strength of concrete.
NZS 3112:3	Methods for test on hardened concrete other than strength
NZS 3114	Concrete surface finish
NZS 3121	Water and Aggregate for concrete

The Contractor shall supply the Engineer with the lean concrete mix design. The mix shall meet the following requirements:

- The maximum aggregate size shall be 37.5 mm.
- Maximum characteristic compressive strength of 20 MPa at 28 days
- The concrete to be used shall be minimum 25% less embodied carbon emission to conventional concrete with all other physical characteristic to remain as per standard concrete
- Nominated Mix. NZS 3104:2003 shall apply. Comprehensive details of the proposed concrete mix shall be submitted to the Engineer at least five working days prior to commencing production acceptance of the mix shall be a hold point. The Engineer will consider the submitted documents prior to authoring release of the hold point.
- Production. The concrete shall be produced from a Ready-Mixed Concrete plant in terms of NZS 3104: 2003. Production and testing shall comply with NZS 3104: 2003.
- Paving Concrete. Paving is to be by fixed form. If hand placing is necessary, concrete shall be deposited and spread uniformly in forms without segregation. The concrete shall be compacted by immersion vibrators and by at least two passes of a hand-guided vibratory screed traversing the full width of the slab on each pass. A suitable head of concrete shall be maintained in front of the screed over its whole length to ensure the uniform transmission of vibration into the slab.

Concrete to be laid on the existing or imported granular subbase at least 100mm.

A polythene sheet could be considered, placed under the concrete subbase to separate it from the underlying subgrade. In soft subgrades (CBR <3.5%) a Class C geotextile should be used under the concrete subbase.

The subgrade at the time of placing the concrete must be clean, free of loose or foreign matter and must not have ponded water. A debonding agent is not required on the subgrade.

Compaction of the concrete shall be through internal vibrators. The quality plan shall detail how the compaction methodology will achieve a homogeneous slab with uniform compaction. A relative compaction of at least 98% is expected. The texturing of the surface shall be by tinning by transverse brooming to provide a U5 texture to NZS 3114: 1987. Do not allow access to vehicular traffic until the concrete has reached an *in-situ* strength 10MPa.

- Edges - Do not saw cut the pavement except the minimum for joint sealing.
- Joints - Saw-cut induced joint to at least 40 mm deep to be provided with joint spacing not more than 4m apart.
- Outer edges - In plan the outer edges must match up to the kerb and channel.
- Special slabs - Odd-shaped slabs of width less than 1.5 metres between joints shall be reinforced with one layer of 663L reinforcing mesh or use steel fibre reinforced concrete. Details shall be included in the Project Quality Plan. Alternatively, if overlaying with asphalt, a suitable paving grid could be placed within the asphalt.
- Geometry and Thickness - The minimum lean mix concrete thickness shall be the design thickness, and the final level shall not result in a reduction in the asphalt thickness requirements.
- Removal and Replacement - If there is non-conforming work Sections 5.6 and 5.7 of NSW R83 shall apply.
- Testing - NZS 3104: 2003 and NZS 3109: 1997 shall apply to concrete testing.

The Contractor must carry out concrete mix design to produce a continuous combined aggregate particle size distribution, consistent with achieving the test properties and other requirements listed above.

The curing compound shall be a water-borne bitumen with the base bitumen being a Grade 180/200 to NZTA M1 and in an emulsion. Alternatively, keeping the surface wet while curing may be acceptable.

The curing compound shall have a residual bitumen content of 0.3 to 0.5 Lm^{-2} although keeping the surface wet while curing may also be considered.

Microcracking might also be considered using a single static pass or an 8-10 tonner roller after 24 hours of curing. However, this approach should consider the long-term effect on the subbase strength.

Details of the proposed mix must be submitted at least ten (10) working days prior to the commencement of the works.

11.8 PAVEMENT JOINT CONSIDERATIONS AND TREATMENT

The design of joints for rigid (concrete) pavements is an integral consideration in their design and must conform to the requirements of Section 6.3.9 of this TDM.

For flexible pavements, joints can occur at various locations within the pavement, including:

- at the interface of new works with existing pavements, which may be transverse, diagonal, longitudinal or other shaped joints, depending on the alignment of the existing and new pavements;
- at the edges and ends of a construction run, e.g. at the end of a production shift within the same pavement configuration;
- at changes in pavement configuration; and
- at the interface with other road elements such as kerb and gutter, subsurface drainage, etc.

The structural competency of flexible pavements at construction joints is generally not as sound as in other areas. The joint is discontinuous in the pavement, forming a plane of weakness, and the adjacent pavement materials tend to be weaker and more permeable.

In addition, stiffness differences between different pavement types can contribute to differential deflections and performance issues. This creates a significant risk of deformation, cracking, and other distress at joints, which is substantially increased where the joint will be trafficked.

The location of joints and the design of joint details to mitigate performance risks is therefore an important part of pavement design.

Joints must not be in wheel paths unless unavoidable. Where unavoidable, joints in wheel paths must be detailed appropriately. Such details can include stepping layer terminations and / or using reinforcing geofabrics and geogrids, as appropriate to the pavement configurations and materials, support conditions, traffic loads, project scope and other factors.

Joint details are required for all pavement joints in a project, including:

- each combination of unique abutting pavement types, which includes joints between new and existing pavements at the extent of works;
- edge and end joints at the end of construction runs within the same configuration; and
- joints between pavements and other road elements, such as kerb and gutter, central medians and subsurface drainage.

These joint details must be shown on the project drawings.

In addition, the following minimum requirements apply to pavement treatment extents from consideration of joining and other issues, unless prior approval from the Department has been obtained:

- Pavement treatments shall extend to the edge of lanes to avoid joints in the wheel paths.
- The pavement treatment shall extend to the limit of geometric changes.
- The extent of pavement treatments must match the extent of pavement marking (traffic control layout) changes, to ensure that all existing pavement marking is removed.

11.9 SURFACING/WEARING COURSE

This specification includes the following:

- The application of the membrane seal upon the finished surface of compacted granular basecourse pavement.
- Construction of one or more layers of asphalt concrete
- Supply of all materials, tools, plant, labour, and supervision necessary for the completion of the whole work in accordance with the contract specification.
- All necessary seal designs and job mix formulae to be completed by the Contractor and approved by the Engineer prior to placing of and form of sealing on the prepared basecourse surface.

11.9.1 Chipseal/ Membrane seal/ SAMI seal

Chipseal design shall be as per NZTA Chipsealing in New Zealand Guidelines and carried out by the contractor and submitted to the Engineer for approval.

A first coat chipseal or membrane seal shall be placed on the finished areas as a protective layer once the granular basecourse construction has been finalised.

Residual application rate of binder shall be minimum 1.4 litre/m2 to be confirmed following by visual inspection and texture testing on site and calculated by the sealing contractor. This may require approval by the Engineer prior to sealing.

The second coat seal be applied in the following sealing season or no later than 12 months after the application of the first coat seal to further protect the surface from water infiltration. However, if construction traffic from the residential development is likely to be concentrated in a particular area, then the two-coat chip seal or temporary seal shall be applied as the initial sealing stage.

If required a Stress Alleviating Membrane Interlay Seal (SAMI Seal) shall be designed with a two coat or racked in seal with a 5% SBS or similar polymer added to the binder.

Refer to ITP for relevant specification, inspection and testing, and acceptance criteria.

11.9.2 Asphalt

Asphalt shall meet the requirements of the NZTA M10 (2020) specification. The asphalt bitumen shall meet the NZTA M01-A (2022) specification.

If required a Stress Alleviating Membrane Interlay Seal (SAMI Seal) shall be designed with a two coat or racked in seal with a 5% SBS polymer added to the binder to mitigate reflective cracking from the underlying layers such as an existing or new CTB treated basecourse with high cement content.

The binder performance grade shall be PG64 16. The binder grade category shall be determined by the traffic environment as detailed in M01-A, however, it is advised to adopt the binder grade category for AT network as per detailed provided in Table 24: Performing Binder Grade & Recommended Mixes. It should be noted sub-surfaces asphalt layers are defined under NZTA M01-A as layers deeper than 80mm below the pavement surface.

For M3 pavements, the performing grade binder to be Heavy (H), Very Heavy (V) or Extreme (E) depending on the traffic loading. All M3 intersections and roundabouts are to have as a minimum Very Heavy (V) binder and an Extreme (E) binder where appropriate.

The design or the Engineer is highly recommended to obtain evidence that the M10 and M01-A requirements have been met, from the asphalt supplier. The Contractor shall provide the information required by Section 3.6 of NZTA M10 (2020) as evidence that the proposed mix design is valid for a particular site. The testing requirements will have been performed by an IANZ approved laboratory. This information will be required for the Engineering Application to minimise the risk of project delay.

Usually a sufficient structural asphalt depth (minimum of 175mm) is to be provided to prevent cracking of cemented materials reflecting to the surface. Geosynthetic membrane should be used for mitigation of reflective cracking on the surface when the total asphalt thickness is less than 200mm paved over the lean-mix concrete or cemented subbase layer

11.10 ROUGHNESS

Acceptable maximum roughness in NAASRA counts/km for a 100-metre section after 12 months of traffic shall not exceed 70 NAASRA counts/km and an average over the project length 60 NAASRA counts/km. This roughness should exclude areas of bridge joints, side road intersections, and existing utility services covers.

11.11 POLISHED STONE VALUE REQUIREMENTS

The Polished Stone Value (PSV) of the aggregates to be used for all surfacing in AT network shall be 55 minimum and shall meet NZTA T10 requirements for the site category. The aggregate shall be selected using the aggregate performance method as provided in Section 8.5

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12.0 CONSTRUCTION CONSIDERATIONS

12.1 CONSTRUCTION ON PEAT SUBGRADES

Peat soils are characterised by low stiffness and are highly moisture sensitive, peats are prone to volumetric changes that are associated with moisture variations and settlement, or expansion can occur over time. The low stiffness means that compaction of overlying layers can mean the peat becomes damaged resulting in lower strength and reduction in stiffness of the peat layers. Ideally peat materials should be removed from the subgrade, however, if this is not possible an additional 300 mm (minimum) of geotextile wrapped dune sand[4], for example Woodhill Sand, shall be placed as a subgrade improvement layer. The 300mm of dune sand shall not be considered as part of the pavement design. If dune sand is not available, then a 250mm of AP65 on both geotextile and geogrid layers shall be placed as the subgrade improvement layer. Compaction of the subgrade improvement layer shall not overstress the peat materials.

Given the moisture sensitivity of peat, care should also be taken to control moisture changes in the peat, otherwise there is a risk of large volumetric changes. Placing subsoil drains in peat material should be considered carefully as this could change the equilibrium water content in the peat resulting in subgrade volume change and potentially cause shape loss in the pavement, therefore subsoil drains shall be either shallowed up, or lined with sealed HDPE, to avoid moisture transfer from or to peat layers.

Typical pavement details developed by legacy Papakura District Council are provided in Appendix E.

12.2 PAVEMENT WIDENING

Pavement widening can result in a discontinuity of materials in the interface between the old and the new pavement in the carriageway. These discontinuities frequently cause localised pavement failure.

The New Zealand Guide to Pavement Structural Design (NZTA) indicates that the discontinuity can be attributed to several factors, in particular segregation of the new aggregate; reduced layer stiffness because of removing the lateral restraint provided by the shoulder; and difficulties associated with compacting layers with a narrow or irregular shape.

The Guide's advice on widening should be followed, particularly about ensuring that there is homogeneity of base materials across the widening interface, stepping interfaces and keeping any interfaces away from wheel paths should all be followed.

Drainage from the existing to the widened areas, when the widened depths may be greater than the existing pavement depths, should be considered. Moisture should have a pathway to drain from the existing pavement.

A minimum widening width usually recommended a lane with or 3.0m minimum should be stipulated sufficient to allow appropriate construction plant to operate and cover the full width of the new lane in consideration that no jointing is in the wheel path. In addition, rreconstruction of parts of the existing pavement may be required to accommodate the widths necessary for equipment to compact materials to specification standards. Tapering down of widening is not allowed.

Widening design should also consider the impact the widening has on street furniture, pathway, boundaries and drainage, particularly where swales are in use.

12.3 CONSTRUCTION WITH SERVICES

Shallow services may be encountered during construction of pavement renewals. Depths of existing services established through GPR survey need to be validated through pot holing or another suitable method like trenching. A construction methodology above these services is to be discussed prior to construction. Potential

[4] Dune sand is uniformly graded which allows for effective compaction with tracked machines or light rollers.

concrete capping of shallow services is also to be discussed with service providers. Should any unanticipated services be found to be affected by the pavement or pavement construction the Engineer should be contacted to agree a suitable solution.

12.4 DRAINAGE

Most subgrade soils in Auckland are clay and silt subgrades which are sensitive to moisture. When saturated they have a reduced bearing shear strength. Severe rainfall events and high-water table levels pose a significant risk to pavement durability at these locations, and it is recommended TNZ F/2 subsoil drainage is provided throughout.

Shoulders should be shaped to direct water away from the pavement. In areas of cut, placement of subsoils, meeting the Auckland Transport requirements is necessary. This will allow removal of excess water from the subgrade and pavement layers.

Subsoil drainage shall also be provided under all new kerb and channel sections and tie into existing drainage systems.

12.5 STRESS ALLIVIATING GEOSYNTHETIC MEMBRANE

Where potential reflective cracking is possible from cement bound layers including concrete subbase layer, a stress alleviating membrane interface (SAMI) layer is recommended between the structural asphalt and the cement bound layers to minimise reflective cracking risk in the pavement.

Geosynthetics can be used to increase the fatigue life of the AC layer by reducing the tensile strain. Research results have demonstrated that the presence of geosynthetics within asphaltic layers can help reduce the magnitude of elastic and plastic asphalt strains, pavement stresses, and surface deflections.

For example, geosynthetic membrane such as Sami-grid or HaTlite XP directly over the cemented materials or substrate or HaTelit or similar industry approved geosynthetic membrane in between two asphalt layers can be used as per the manufacturer's specification to reduce the risk of reflective cracking in the overlaying layers.

12.6 MICROCRACKING OR PRE-CRACKED STATE

The use of heavily cemented basecourse aggregate materials is not permitted as a general treatment. This is because a bound basecourse is likely to create cracking of the pavement as the material cures and induces shrinkage cracking. However, there may be situations where other construction methodologies cannot be used, in this case, the use of bound basecourse materials should be approved by Auckland Transport after being supplied with the justification for this high-risk treatment.

To mitigate large shrinkage cracks the construction team should implement micro-cracking or pre-cracking and geosynthetic crack inhibitors as above to be applied as per the manufacturers' specification. Care should be taken not to completely break down the cemented bonds. In cement bound basecourse materials this could include the use of trafficking the layer with a steel drum roller on highest amplitude lowest vibe between 24 hours and 48 hours after construction. The number of passes should be considered as well, for example, three passes in each direction, is recommended to induce micro-cracks at relatively close spacing.

12.7 GEOTEXTILE

At an unexpected event of under cutting to subgrade level a Class C geotextile separation or similar is recommended to be laid at the subgrade level prior to construction of new pavement to prevent contamination. Similarly, on subgrades with a CBR less than or equal to 3.5% a Class C geotextile separation layer shall be used to prevent contamination of the pavement materials with fine materials.

12.8 WEATHER

Construction should be avoided during winter where wetter months are expected to be more frequent. The success of a stabilised granular pavement and surface construction is highly dependent on the moisture condition of the various layers, especially prior to sealing.

Pavement construction should be limited to the accepted summer construction season, and it is expected to be difficult on weaker subgrades in wet weather due to the potential for subgrades to remould to a weaker state under loading, their potential expansive nature, and the high sensitivity of some materials to moisture.

To ensure that ground water effects during construction are minimised it is recommended that construction programming account for long range weather forecasts and allow for the spatial and unpredictable nature of Auckland rainfall events by having construction programmes that allow work to cease during wet weather periods. Historically, the summer pavement construction season in Auckland is usually between the mid to end of September through to March.

12.9 TRAFFIC MANAGEMENT

To be maintained to an acceptable level of service to the public, no construction work should be permitted outside the hours stated in the Traffic Management Plan (TMP).

NZ Transport Agency, with input from WorkSafe, construction and maintenance suppliers and the road safety sector, has published new best practice guidance to improve safety at work sites – a key challenge facing our transport system in 2024.

This guide to temporary traffic management outlines how to use a risk-based approach to plan and mitigate the risks to road workers and road users to keep them safe. The guide provides advice to organisations on how to put risk assessment and planning first before decisions on control types and equipment are made. The new risk-based approach ensures that Temporary Traffic Management (TTM) setups are as safe as possible for the specific risks at each site. It will be useful for road construction and maintenance, events, emergency response or any activity where a temporary road design is required.

AT has adopted this version and prepared TTM based on this risk-based approach and NZTA guidelines.

13.0 REFERENCES

1. NZTA New Zealand Guide to Pavement Evaluation and Treatment Design, 2018
2. NZTA New Zealand Guide to Pavement Structural Design, 2018
3. NZTA Asphalt surfacing treatment selection guidelines 2013 Amendment 2.1
4. #17-01 Asphalt Depth at high stress locations for new pavements and renewals – Waka Kotahi (2017) https://www.nzta.govt.nz/resources/17-01-asphalt-depths-at-high-stress-locations-for-new-pavements-and-renewals/
5. Roads and Streets Frameworks, Auckland Transport 2020 - https://at.govt.nz/media/1983549/roads-and-streets-framework-may-2020-web.pdf
6. Stormwater Code of Practice (SWCoP) (Chapter 4) Auckland Council's Code of Practice for Land Development and Subdivision - https://www.aucklanddesignmanual.co.nz/content/dam/adm/adm-website/developing-infrastructure/infrastructure-codes-of-practice/chapter-4-stormwater/CoP_Chapter_4_v4.pdf
7. Austroads Guide to Pavement Technology Series, Sydney, Australia, soe of the referred documents as follows:
   - Part 2: Pavement Structural Design (AGPT-2) 2017 & 2024 (recent release)
   - Part 3: Pavement Surfacings 2021
   - Part 4: Pavement Materials 2007
   - Part 4A: Granular Base and Subbase Materials 2024
   - Part 4B: Asphalt 2014
   - Part 4F: Bituminous Binders 2017
   - Part 5: Pavement Evaluation and Treatment Design 2019
8. Austroads Technical Specification ATS-3466-23 High Friction Surface Treatment 2023
9. Austroads Guide to Traffic Management Part 12: Integrated Transport Assessments for Developments 2020
10. Guidelines and Procedures to Traffic Generating Developments, Sydney Roads and Traffic Authority (RTA), Transport Planning Section, 2002 Issue 2.2
11. RIGID PAVEMENT – Volume CJ - Jointed Reinforced Concrete Pavement (Sheet 1 to 15), Standard Details Road & Maritime Services, NSW Government Transport 2015.
12. Guidelines for 1993 AASHTO Pavement Design, Virginia Department of Transportation, Pavement Design and Evaluation Section 2003

APPENDICES

Appendix A
EXAMPLE INSPECTION AND TEST PLAN

An example Inspection and Test Plan for chipsealing is displayed in Table 32. Generic inspection and test plans have been developed for the pavement and surfacing designs contained in the catalogue design charts to allow builders to easily identify what quality assurance testing is required prior, during and after construction. However, these inspection and test plans will be minimum requirements, and the contractor should add additional hold points and test requirements to manage the construction risks.

The full inspection and test plan is available as an excel spreadsheet, separate to this document.

Table 32: Inspection and Test Plan for Unbound Granular Pavement with Thin Asphalt or Chip Seal Surfacing

Project Name:	**Location:**
ITP Prepared By: (Contractor)	**Date:**

No.	Activity Description	Verification Activity	Methods or reference	Frequency	Acceptance criteria	Type	Records
Subgrade: The subgrade shall be prepared in accordance with the requirements of the Geotechnical Report, the Project Specifications, and AT Transport Design Manual.							
1	Excavated subgrade	No unacceptable subgrade materials after excavation	Visual inspection	NA	Check records of site inspection and decision making.	Construction Testing	Site diary.
2	Subgrade preparation, subgrade stabilisation, Scala Test & Proof Roll or Clegg Hammer Impact tests (in the case of non-cohesive subgrade materials).	Subgrade meets the design strength.	NZS 4402 :(1988) Test 6.5.2 and the correlation to CBR shall be as per Fig 5.3 Austroads Guide to Technology Part 2: Pavement Structural Design. NZTA B/9 specification for subgrade stabilisation.	1 in each wheel path at 10 m intervals & not < 1.0m depth below subgrade surface, minimum 1 per 10m2 of exposed subgrade or two tests per undercut area, whichever is the greater (i.e. add tests in any shoulder at 1/30m2 frequency) and with any change in material appearance.	**(a): Untreated Subgrade** Proof of subgrade testing & action taking for CBR strength found in accordance with drawings and specifications. Records of undercut depth for CBR found and backfill materials used. Recorded evidence of proof roll to ensure that no deformation and springing is observed under the proof roll. **(b): Stabilized Subgrade** Proof of subgrade testing & action taking for CBR strength found in accordance with drawings and specifications. Check subgrade stabilisation used chemical type and dosage confirmed by soaked CBR lab testing of samples. Check optimum water content & maximum dry density was determined. Check for records of plateau density tests, compaction achieved and surface shape/level meeting NZTA B/9. Proof of Scala Test on the stabilized	Construction Testing and Material Testing	Laboratory testing data, site diary and site testing records.

No.	Activity Description	Verification Activity	Methods or reference	Frequency	Acceptance criteria	Type	Records
					subgrade for subgrade CBR ≥ design CBR is achieved. Evidence of proof roll to ensure that no deformation and springing is observed under the proof roll.		
3	Evidence of pavement design thickness (Stringline Checks or Survey Levels)	Subgrade level is at or below design level.	Level survey or dipping from stringline.	Each design pavement depth every 10m	Measurements to be taken across the carriageway. Check records that subgrade surface is at or below the finished surface level minus the pavement depth.	Construction Testing	Site diary and testing records.
Subsoil							
4	Subsoil drainpipe	Subsoils in accordance with AT TDM KC0007	AT TDM Standard Detail KC0007, NZTA F/2 (2013) Pipe Subsoil Drain Construction		Check records of subsoil drain level start & finish to ensure at least 0.5% grade has been constructed. Check in catchpit that subsoil drain connection is at or above the sofit level of the outlet pipe. Check evidence of using subsoil drainage aggregate - dockets showing TNZ F/2 filter material used on site.	Construction Testing and Material Testing	Site diary and delivery dockets.
Unbound Granular Subbase							
5	Material Supply	AT TDM Specifications	Subbase supplied to meet AT TDM Specification for Aggregate Supply	As per AT TDM	Check aggregate supplied complies with TDM Section 3: Specifications for Transport Infrastructure, Series 0800 - Specification for Supply of Aggregates.	Material Testing	Material supply dockets.
6	Compaction	Density Check	Subbase course construction as per NZTA B/2 (2005)	As per NZTA B/2	Check compliance with NZTA B/2 compaction requirements for subbase course.	Construction Testing	Density testing records.

No.	Activity Description	Verification Activity	Methods or reference	Frequency	Acceptance criteria	Type	Records
7	Level Checks / String Line	Finished Level Check	Subbase construction as per NZTA B/2 (2005)	Every 10m	Measurements to be taken across the carriageway. Check records that subbase course surface is at or below the finished surface level minus the surfacing and basecourse depth.	Construction Testing	Site diary and testing records.
Unbound Granular Basecourse							
8	Material Supply	AT TDM Specifications	Basecourse supplied to meet AT TDM Specification for Aggregate Supply	As per AT TDM	Check aggregate supplied complies with TDM Section 3: Specifications for Transport Infrastructure, Series 0800 - Specification for Supply of Aggregates.	Material Testing	Material supply dockets.
9	Compaction	Density Check	Basecourse construction as per NZTA B/2 (2005)	As per NZTA B/2	Check compliance with NZTA B/2 compaction requirements for basecourse.	Construction Testing	Density testing records.
10	Level Checks / String Line	Finished Level Check	Basecourse construction as per NZTA B/2 (2005)	Every 10m	Measurements to be taken across the carriageway. Check records that basecourse surface is at the finished surface level within the tolerances allowed in NZTA B/2.	Construction Testing	Site diary and testing records.
11	Surface Preparation	Visual Check	Chipsealing in New Zealand (2005)	Continuous	Check evidence that surface was tight and free of cakes of fines or loose aggregate material and the surface is tight.	Construction Testing	Site diary, photographs
12	Deflection (Benkelman Beam Test)	Pavement Strength Check	AT TDM for road hierarchy and as shown on the Drawings	Every 10m staggered in wheelpath	Check deflection testing is within limits required in AT TDM for road hierarchy and as shown on the Drawings.	Construction Testing	Beam test results.
13	Degree of Saturation	Moisture Check	Basecourse construction as per NZTA B/2 (2005)	As per NZTA B/2	Check records of degree of saturation of basecourse before sealing	Construction Testing	Site diary and testing records.
Wearing Surfaces							
14	Chip Seal or Membrane Seal	Chip Sealing in accordance with design	Chipsealing in NZ (2005)	Continuous	Check the designed type and grade of chip seal or membrane seal (under asphalt) has been constructed in accordance with specified application rates or alternatively Contractor's calculated rates - minimum residual bitumen has been sprayed and correct chip size(s) applied.	Construction Testing	QA records for sealing.

No.	Activity Description	Verification Activity	Methods or reference	Frequency	Acceptance criteria	Type	Records
15	Chip Seal or Membrane Seal Materials	Materials meet specifications	NZTA M6 specification for chip NZTA M01-S for bitumen binder	Each seal type	Check the materials comply with the standard specifications	Materials Testing	Testing records
16	Asphalt Design	Mix Design check	NZTA M/10 specification	Each mix used	Check the asphalt mix has been designed in accordance with NZTA M/10 and the mix design report is not greater than 2 years old in accordance with M/10.	Laboratory Design Materials Testing	Asphalt QA
17	Skid Resistance	Mix Design check	NZTA T/10 skid resistance specification	Each mix used	Check coarse aggregate used meets aggregate performance history for Site Category, check mix design for this. Design for project roads should specify when T/10 skid resistance applies and what Site Category is applied.	Laboratory Design Materials Testing	Asphalt QA
18	Asphalt Production Tests	Asphalt meets Mix Design	NZTA M/10 specification	As per NZTA M/10	Check production Grading, Binder Content, Max Density, Air Voids, meet NZTA M/10 requirements.	Laboratory Testing	Asphalt QA
19	Weather Limitations	Air Temperature and rain check	Project Specification, NZTA M10 specification	Every day	Check the asphalt was not paved when foggy or raining or placed on a wet surface. Check the air temperature was greater than the minimums in NZTA M/10.	Construction Inspection	Site diary, paving records
20	Jointing	Joint Arrangements	NZTA M/10 specification, Drawings, Project Specification.	All joints	Check Paving Plan to be completed for the site with dimensions. Check joint sealing applied between new and existing surfacing. Check joints should be offset by minimum 150mm and as per Drawings.	Construction Inspection	Paving Plan
21	Asphalt thickness	Thickness check	Drawings, Project Specification, NZTA M/10 Specification.	Continuous	Check thickness of asphalt checked either by coring or level check.	Construction Testing	Laboratory report, site checks.
22	Air Voids	Air Void Check	NZTA M/10 specification	As per NZTA M/10	Check core air voids comply with NZTA M/10 characteristic value limits. If air voids not taken check correlation between density testing and air voids used for determining compaction.	Construction Testing	Laboratory report, or site records.

No.	Activity Description	Verification Activity	Methods or reference	Frequency	Acceptance criteria	Type	Records
23	Roughness	Surface Finish.	Contract Specification, Drawings or AT TDM	Continuous	If roughness testing required, check average NAASRA Count complies with specification.	Construction Testing	Roughness report.
24	Final Surface Shape and Tolerances	Surface Finish.	NZTA M/10 specification	Continuous and at joints	Check surface shape measurements are in accordance with NZTA M/10.	Construction Inspection	Scala sheet Pavement log String Line Sheet
25	Roadmaking	Visual	NZTA M7 approved (Class B or C as required for anticipated traffic level)	Continuous	As per design Drawings and Project Specification.	Construction Inspection	Product used data sheets.

Approved by:		Signature:		Date:	

Table 33: Example Inspection and Test Plan for Field / Laboratory Testing Chip Seal Surfacing

Element	Requirement	Evidence	Test Reference	IANZ	Test Frequency	Acceptance Criteria	Hold Point	Responsibility
Seal design	Seal design documented and reviewed	Signed review	N/A	N/A		Signed design and signed review	Hold	
	Chips meet M6	Test report	In M6	Yes		Satisfactory test results	Hold	
Source properties	Crushing resistance natural aggregates	Test results	NZS 4407 Test 3.10	Yes	CCNZ BPG05:2019	Less than 10% fines generated under 230 kN load	Hold	
	Crushing resistance of melter aggregate	Test results	NZS 4407 Test 3.10	Yes	CCNZ BPG05:2019	Less than 13% fines generated under 230 kN load	Hold	
	Weathering resistance	Test results	NZS 4407 Test 3.11	Yes	CCNZ BPG05:2019	Quality index AA or BA	Hold	
	Weak particles test	Test results	AS 1141.32-1995	Yes	CCNZ BPG05:2019	Maximum of 1% of unsound materials	Hold	
	Polished stone value, grades 2,3, and 4	Test results	BS EN 1097-8:2009	Yes	CCNZ BPG05:2019	According to T10 or as detailed in contract requirements	Hold	
	Polished stone value, grades 5 and 6	Test results	BS EN 1097-8:2009	Yes	CCNZ BPG05:2019	According to T10 or as detailed in contract requirements	Hold	
Production properties	Cleanness Grade 2	Test results	NZS 4407:1991 Test 3.9	Yes	CCNZ BPG05:2019	>89	Hold	
	Cleanness Grade 3	Test results	NZS 4407:1991 Test 3.9	Yes	CCNZ BPG05:2019	>87	Hold	
	Cleanness Grade 4	Test results	NZS 4407:1991 Test 3.9	Yes	CCNZ BPG05:2019	>85	Hold	
	Cleanness Grade 5 and 6	Test results	NZS 4407:1991 Test 3.9	Yes	CCNZ BPG05:2019	Contact specifications	Hold	
	Size and shape grades 2, 3, and 4	Test results	NZS 4407:1991 Test 3.13	Yes	CCNZ BPG05:2019	Meets table 2	Hold	
	Grades 5 and 6 clean, tough and free of soft, weathered, disintegrated or deleterious material.	Test results	None	Not applicable	Not applicable	None	Hold	
	Grading of Grades 5 and 6	Test results	NZS 4407:1991 Test 3.8	Yes	CCNZ BPG05:2019	Meets table 3	Hold	

Element	Requirement	Evidence	Test Reference	IANZ	Test Frequency	Acceptance Criteria	Hold Point	Responsibility
Binder	Bitumen meets M1-A	Test report	In performance grade test M1-A	Yes		Satisfactory test results	Hold	
Production properties	Resistance to foaming	Visual Inspection	n/a		Each shipment	No foaming up to 175 °C	Hold	
	Consistency of sampling methodology	Test Sheets	ASTM D140 (or equivalent)		All tests	Sampling as per ASTM D140 (or equivalent)	Hold	
	QA Plan - NZTA approval	Written approval	n/a		Per bitumen source, composition or production process (max approval duration of 5 year).	NZ TA review & approval	Hold	
	Manufacturing plant certification	Certification	Certification to ISO 9001 (or equivalent)		Per bitumen source, composition or production process (max approval duration of 5 year).	Certification to ISO 9001 (or equivalent)	Hold	
Bitumen Properties (Emulsion)	Residual Binder Extraction from Emulsions for Quality Assurance Testing NZ Transport Agency Research Project October 2008 Research Report 360; Tests include but not limited to residue by evaporation test, sieve test, viscosity test, distillation test, storage stability test, density test, particle charge test etc.							
Bitumen Properties (Cutback)	Binder hardness	Test Sheets	ASTM D5 Penetration at 25 °C, 100 g, 5 s	Yes	Each shipment or production batch	Grade 80 - 100: 80 ≤ Pen ≤ 100 Grade 130 - 150: 130 ≤ Pen ≤ 150 Grade 180 - 200: 180 ≤ Pen ≤ 200	Hold	
	Binder flow behaviour - Viscosity @ 60 °C	Test Sheets	AS 2341.2 or ASTM D2171 Viscosity (Pas) @ 60 °C	Yes	Each shipment or production batch	Grade 80 - 100: Viscosity ≥ 115 Grade 130 - 150: Viscosity ≥ 58 Grade 180 - 200: Viscosity ≥ 36	Hold	
	Binder flow behaviour - Viscosity @ 70 °C (only required if Viscosity @ 60 °C criterion is not achieved)	Test Sheets	AS 2341.3 or ASTM D2170 Viscosity (mm2/s) @ 70 °C	Yes	Each shipment or production batch	Grade 80 - 100: Viscosity ≥ 40,000 Grade 130 - 150: Viscosity ≥ 21,000 Grade 180 - 200: Viscosity ≥ 14,000	Hold	
	Binder flow behaviour - Viscosity @ 135 °C	Test Sheets	AS 2341.3 or ASTM D2170 Viscosity (mm2/s) @ 135 °C	Yes	Each shipment or production batch	Grade 80 - 100: 300 ≤ Viscosity ≤ 650 Grade 130 - 150: 190 ≤ Viscosity ≤ 450 Grade 180 - 200: 140 ≤ Viscosity ≤ 350	Hold	

Element	Requirement	Evidence	Test Reference	IANZ	Test Frequency	Acceptance Criteria	Hold Point	Responsibility
	Flash Point -Cleveland open cup test	Test Sheets	ASTM D92 Flash Point (°C)	Yes	Each shipment or production batch	All Grades: Flash Point ≥ 218	Hold	
	Solubility	Test results	ASTM D2042 Solubility in Trichloroethelene	Yes	Each shipment or production batch	All Grades: Solubility ≥ 99.5%	Hold	
			AS 2341.8 Solubility in Toluene	Yes		All Grades: Solubility ≥ 99.0%	Hold	
	Aged bitumen - hardness / ductility of residue from RTFO test (RTFO - ASTM D2872 or AS 2341.10)	Test results	ASTM D5 Penetration at 25 °C, 100 g, 5 s	Yes	Each shipment or production batch	Grade 40 - 50: Pen ≥ 45 All Other Grades: Pen ≥ 50	Hold	
		Test results	ASTM D113 Ductility (m) @ 25 °C	Yes	Each shipment or production batch	All Grades: Ductility ≥ 60	Hold	
	Durability	Test results	TNZ T/13 Durability test value (MPa) @ 5 °C & 9 Hz	Yes	Each shipment or production batch	Grade 40 - 50: ≤ 130 Grade 60 - 70: ≤ 120 All Other Grades: ≤ 100	Hold	
Surface preparation	Pavement surface meets NZTA B2 (unbound)	Photographic	In NZTA B2	No		Meets quality plan	Hold	
	Basecourse layer compaction	Measurement			Minimum of 1 per 5000 square metres	Mean density as percentage of MDD >=98 of MDD BC	Hold	
	Surface finish	Photographic		Photo	Continuous Observation	Large aggregate exposed, held in place with smaller aggregate, smaller aggregate held in place by fines. Matrix does not displace under normal traffic	Hold	
	Preseal requirement	Degree of saturation per lot			5 random test per lot	DOS < 80%	Hold	

Approved by:		**Signature:**		**Date:**	

Appendix B
AT ELECTRIC BUS WEIGHT / MASS DATA

Below information provide a generic 2-axle, 3-axle and double decker electric bus mass / weight with and without passenger traffic to be used for pavement design.

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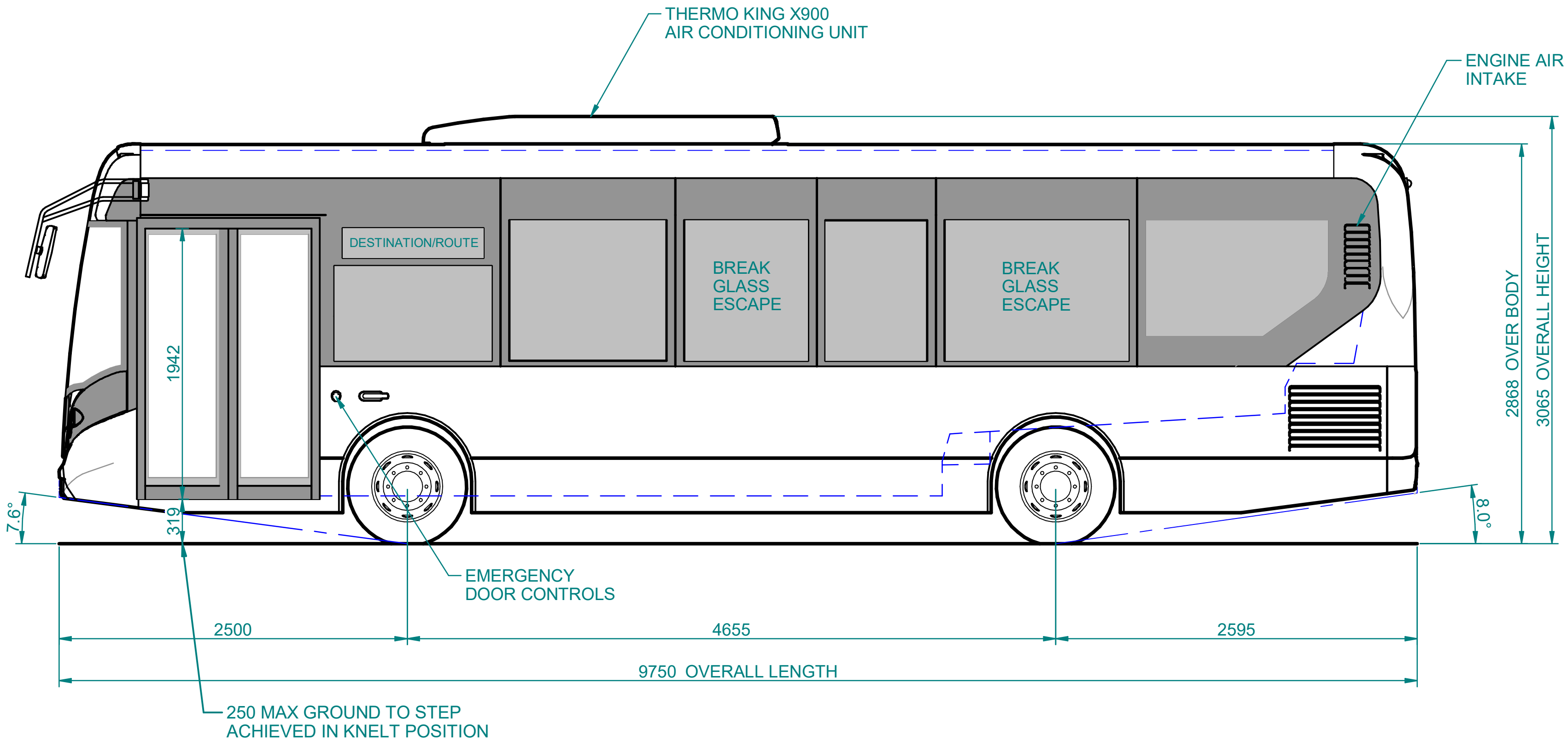


LEFTHAND ELEVATION

ALEXANDER DENNIS

Form No EN-059

Rev	Date	Description	Name
01	21-06-18	Passenger capacity updated following built vehicle weigh reports.	J.Duff

Title :
9.7 METRE MMC ENVIRO 200 BODY
4 CYL DENNIS CHASSIS - (EURO 6)

REF: 'RITCHIES - NEW ZEALAND'

Drawn : J.Duff

Date : 06-03-18

Enq :

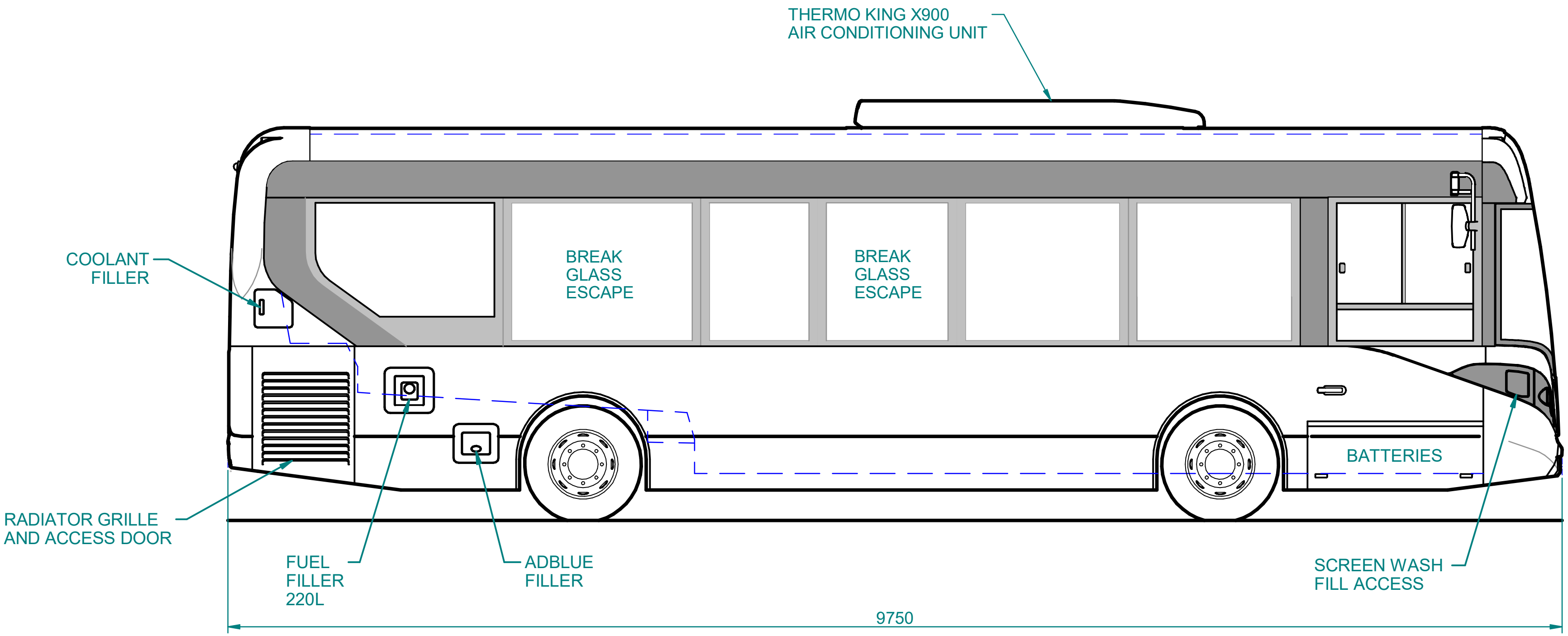
Seat Type :
Lazzerini Pratico
Low Back

Alexander Dennis Ltd
91 Glasgow Road
Falkirk FK1 4JB
T- 01324 621672 F- 01324 621746

Iss No : 01

Page No : 1

Drg No :
H294GA



RIGHTHAND ELEVATION

Rev	Date	Description	Name
01	21-06-18	Passenger capacity updated following built vehicle weigh reports.	J.Duff

ALEXANDER DENNIS

Form No EN-059

Title :
9.7 METRE MMC ENVIRO 200 BODY
4 CYL DENNIS CHASSIS - (EURO 6)

REF: 'RITCHIES - NEW ZEALAND'

Drawn : J.Duff

Date : 06-03-18

Enq :

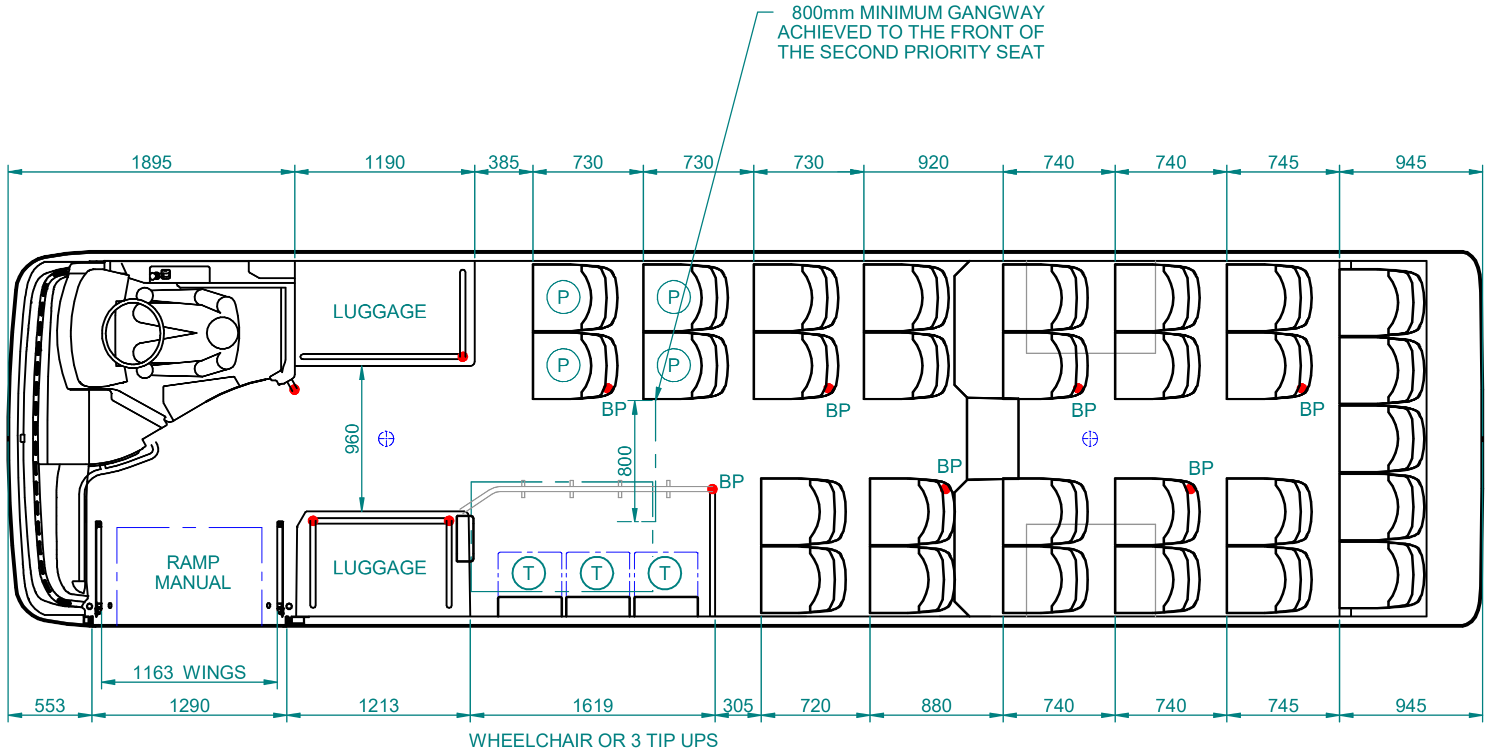
Seat Type :
Lazzerini Pratico
Low Back

Alexander Dennis Ltd
91 Glasgow Road
Falkirk FK1 4JB
T- 01324 621672 F- 01324 621746

Iss No : 01

Page No : 2

Drg No : H294GA



(P) INDICATES PRIORITY SEAT
(T) INDICATES TIP UP SEAT
(BP) INDICATES BELL PUSH

CHASSIS PLATED WEIGHT

FRONT AXLE	5000kg
SECOND AXLE	8800kg
GVW	13800kg

AVERAGE PASSENGER/DRIVER MASS = 80kg
STANDEE AREA = 0.17/MTR SQ

CAPACITY BASED ON 12,000kg MAXIMUM LADEN WEIGHT.
32 SEATED PLUS 17 STANDEES AND 0 WHEELCHAIRS
OR
29 SEATED PLUS 17 STANDEES AND 1 WHEELCHAIR

CAPACITY BASED ON FLOOR AREA (12,000kg LADEN EXCEEDED).
32 SEATED PLUS 21 STANDEES AND 0 WHEELCHAIRS
OR
29 SEATED PLUS 21 STANDEES AND 1 WHEELCHAIR
OR
29 SEATED PLUS 27 STANDEES AND 0 WHEELCHAIRS

ALEXANDER DENNIS

Form No EN-059

Rev	Date	Description	Name
01	21-06-18	Passenger capacity updated following built vehicle weigh reports.	J.Duff

Title :
9.7 METRE MMC ENVIRO 200 BODY
4 CYL DENNIS CHASSIS - (EURO 6)

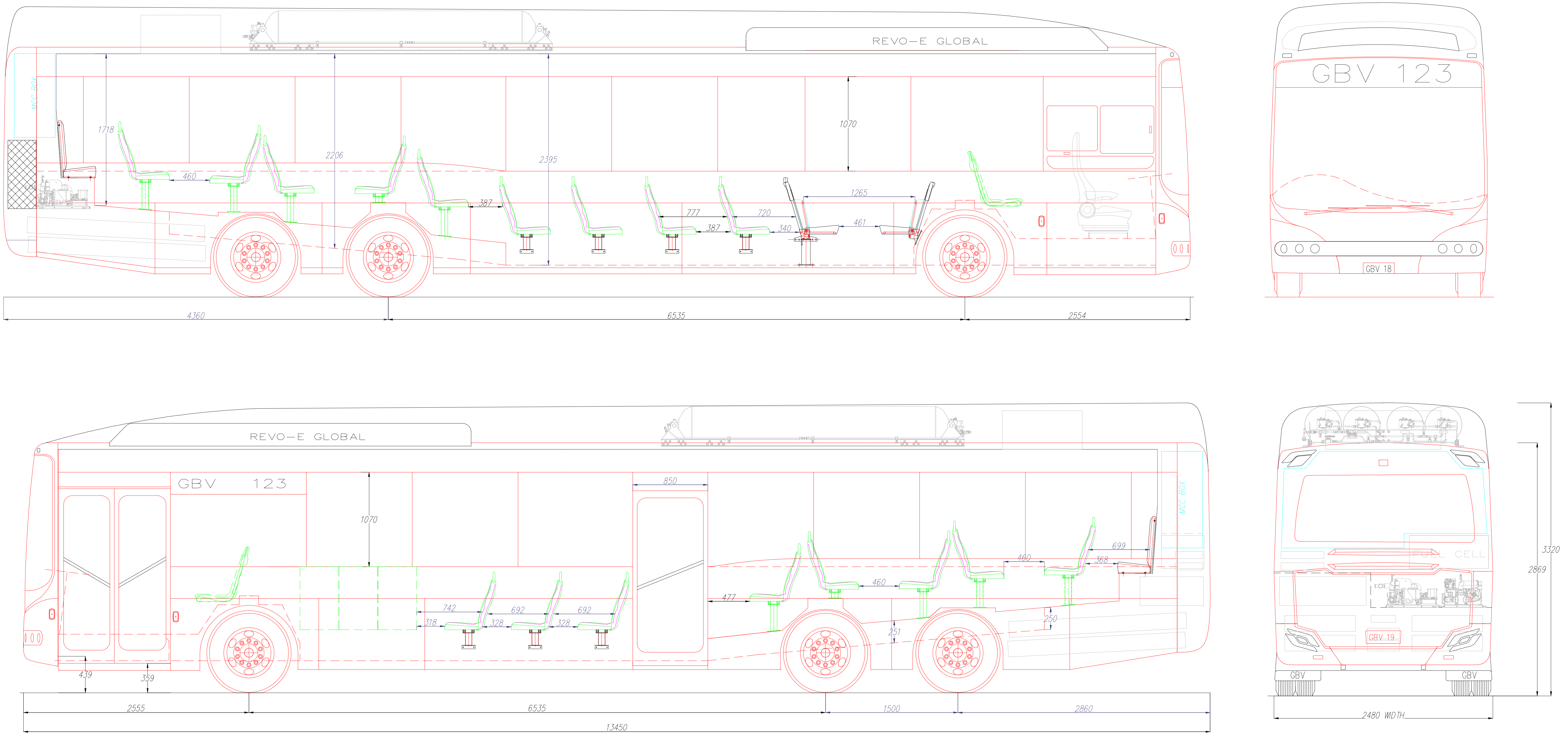
REF: 'RITCHIES - NEW ZEALAND'

Drawn : J.Duff
Date : 06-03-18
Enq :

Seat Type :
Lazzerini Pratico
Low Back

Alexander Dennis Ltd
91 Glasgow Road
Falkirk FK1 4JB
T- 01324 621672 F- 01324 621746

Iss No : 01
Page No : 3
Drg No :
H294GA



AXLE LOADINGS	FRONT	REAR	TOTAL
CHASSIS TARE ESTIMATE	1685	7955	9640
BODY TARE ESTIMATE	1730	2310	4040
TOTAL TARE ESTIMATE	3415	10265	13682
47.PASSENGERS + DRIVER @ 80kg	990	2850	3840
31 STANDING PASSENGERS @ 80kg	1320	1160	2480
GROSS VEHICLE WEIGHT	5725	14275	20000
MANUFACTURES GROSS RATING	6300	19000	25000
NZ MAX.AXLE WEIGHTS	6000	14500	25000
LOAD DISTRIBUTION	29%	71%	100%
CAPACITY OF 275/70R22.5 TYRES	6300	17900	24200

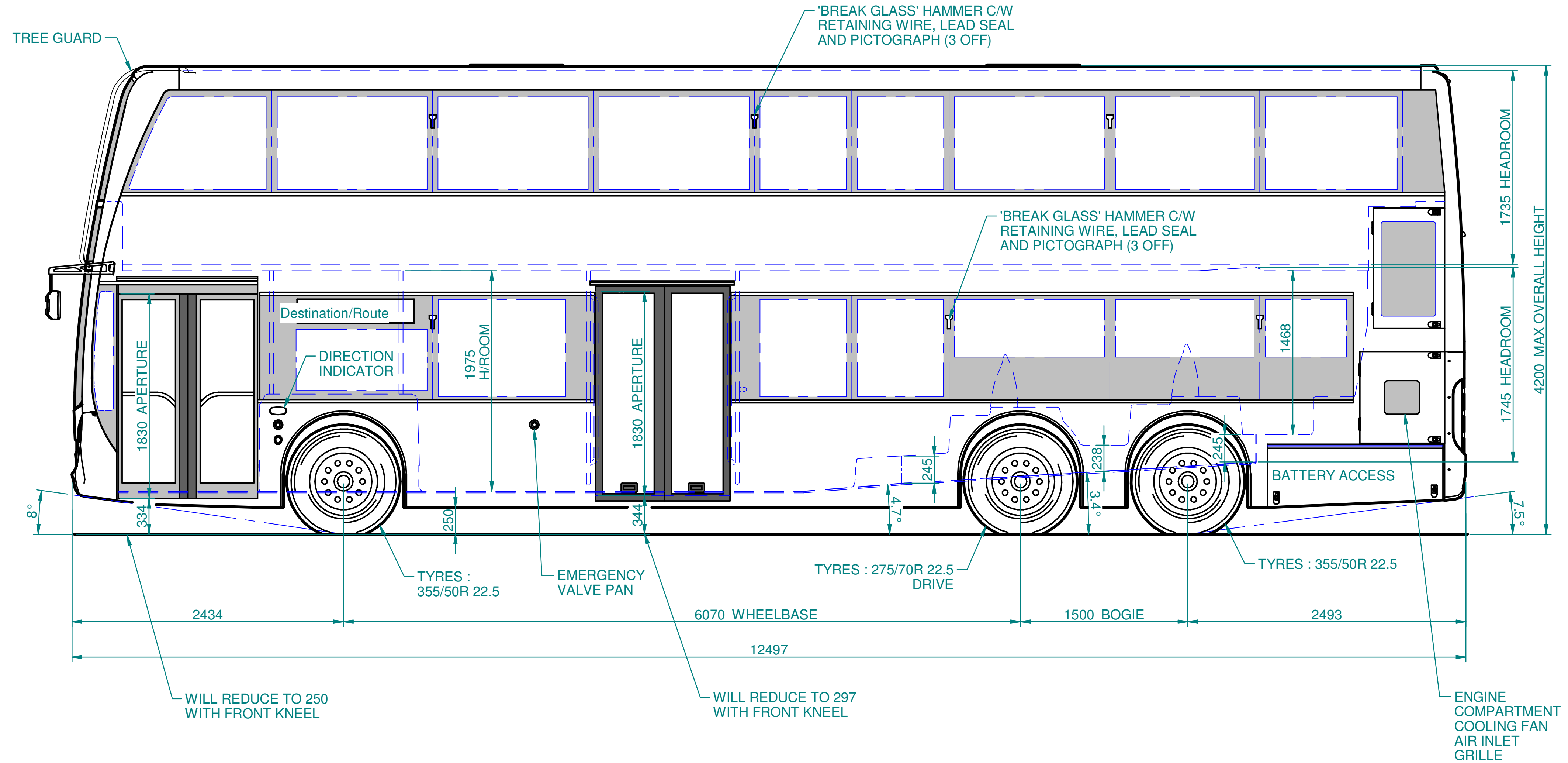
47 SEAT LOW FLOOR COMMUTER BUS
ON GBV 3 AXLE HEV CHASSIS
ALUMINIUM ALLOY FRAME
SEATS: VOGEL 750

OPERATOR: –

Global Bus Ventures.

63 DETROIT DRIVE, PO BOX 104
ROLLESTON 7675, NEW ZEALAND
TEL: +64 03 347 2167
FAX: +64 03 347 9027

SCALE	N.T.S
DATE	31/01/19
CLASS	OMNIBUS
SEATING	47+1
CHASSIS	EV 3 AXLE
MODEL	ENVIROLINE
VENTILATION	SPHEROS
AIR CON.	REVO–E
DWG.NO.	GBV3172



LEFTHAND ELEVATION

ALEXANDER DENNIS

Form No EN-059

Rev	Date	Description	Name
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02	21-05-13	WAS SINGLE EXIT DOOR - SEATING MODIFIED	P.F.
03	15-08-14	SEATING MODIFIED AND SHEET 3A & 4A DEL	S.T
04	22-08-14	STEERED TAG NOTE DELETED FROM SHEET 1	S.T

Title : ENVIRO 500 12.5 METRE BODY
DENNIS E500 OFFSET T-DRIVE CHASSIS
DISC BRAKES AND 'ZF' AV132 DRIVE AXLE

REF : 'NEW ZEALAND'

Drawn : P.F.

Date : 11-04-13

Enq :

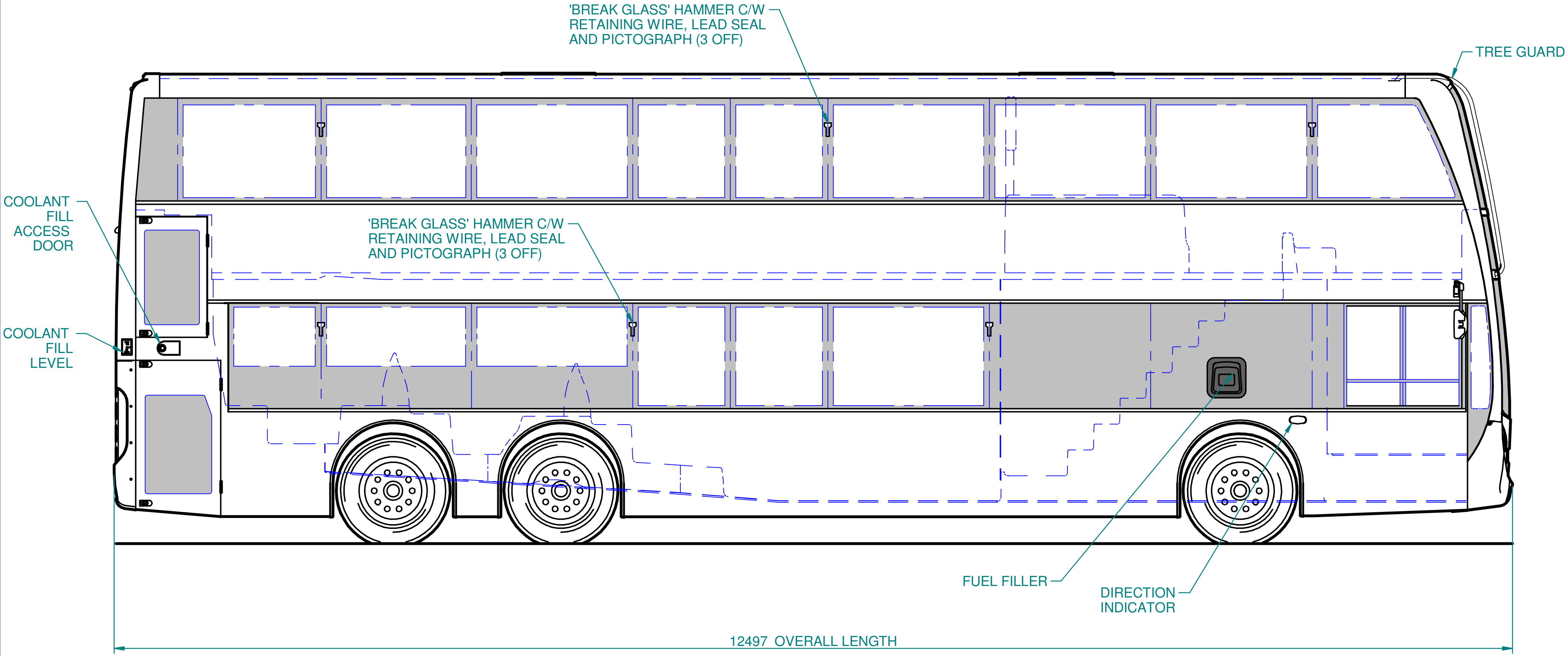
Seat Type :
Practico Lightweight
Low Back & High back L/S
& High back U/S

Alexander Dennis Ltd
91 Glasgow Road
Falkirk FK1 4JB
T- 01324 621672 F- 01324 621746

Iss No : 03

Page No : 1

Drg No : WA4901



RIGHTHAND ELEVATION

ALEXANDER DENNIS

Form No EN-059

Rev	Date	Description	Name
01	18-04-13	RELEASE ISSUE	P.F.
02	21-05-13	WAS SINGLE EXIT DOOR - SEATING MODIFIED	P.F.
03	15-08-14	SEATING MODIFIED AND SHEET 3A & 4A DEL	S.T
04	22-08-14	STEERED TAG NOTE DELETED FROM SHEET 1	S.T

Title : ENVIRO 500 12.5 METRE BODY
DENNIS E500 OFFSET T-DRIVE CHASSIS
DISC BRAKES AND 'ZF' AV132 DRIVE AXLE

REF : 'NEW ZEALAND'

Drawn : P.F.

Date : 11-04-13

Enq :

Seat Type :
Practico Lightweight
Low Back & High back L/S
& High back U/S

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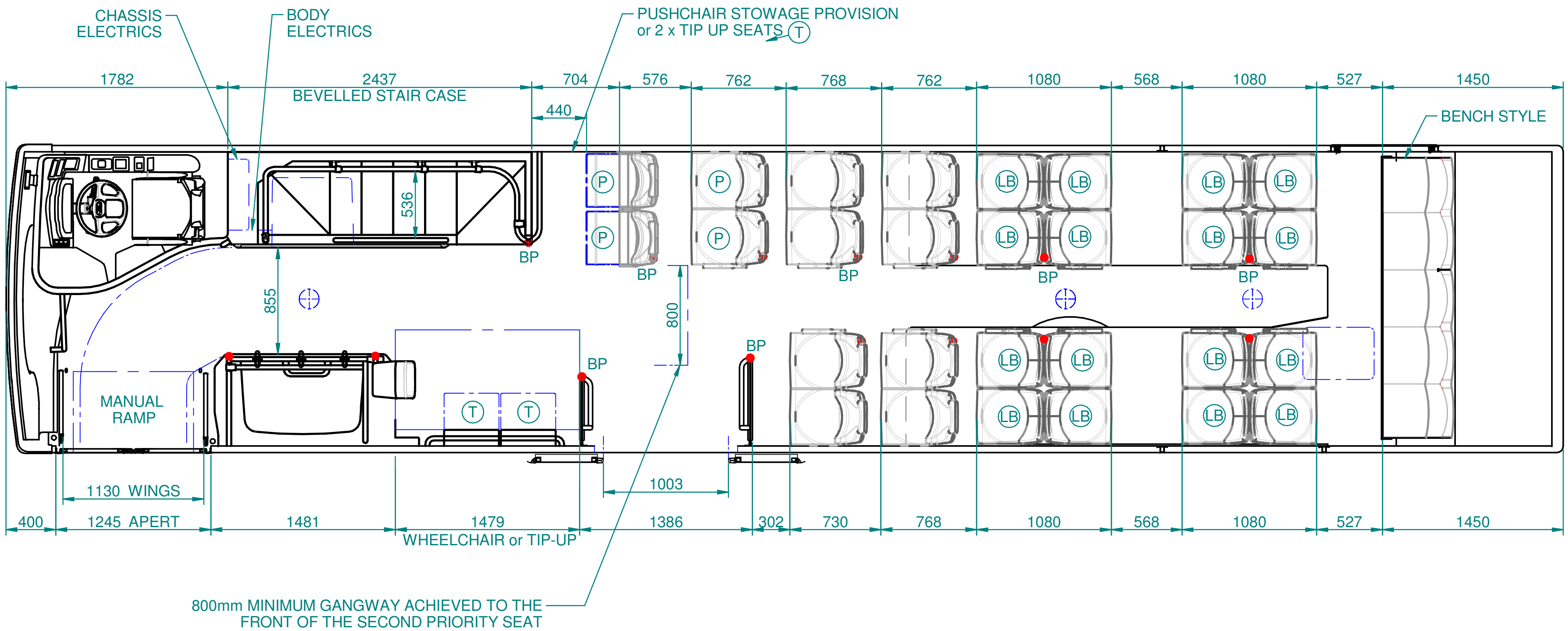
Iss No : 03

Page No : 2

Drg No : WA4901



LOWER SALOON SEATING 35 WITH 0 WHEELCHAIR
PRACTICO LIGHTWEIGHT HIGHBACK DOUBLE 5 + 1 TIP UP
PRACTICO LIGHTWEIGHT LOWBACK DOUBLE 8 BACK TO BACK SEATS
FIVE REAR SEAT BENCH STYLE 1
LOWER SALOON STANDING - SEE WEIGHT DETAILS

BP : BELL PUSH
(T) TIP UP SEATS
(P) PRIORITY SEATS
(LB) PRACTICO LIGHTWEIGHT LOWBACK

ALEXANDER DENNIS

Form No EN-059

Rev	Date	Description	Name
01	18-04-13	RELEASE ISSUE	P.F.
02	21-05-13	WAS SINGLE EXIT DOOR - SEATING MODIFIED	P.F.
03	15-08-14	SEATING MODIFIED AND SHEET 3A & 4A DEL	S.T
04	22-08-14	STEERED TAG NOTE DELETED FROM SHEET 1	S.T

Title : ENVIRO 500 12.5 METRE BODY
DENNIS E500 OFFSET T-DRIVE CHASSIS
DISC BRAKES AND 'ZF' AV132 DRIVE AXLE

REF : 'NEW ZEALAND'

Drawn : P.F.
Date : 11-04-13
Enq :

Seat Type :
Practico Lightweight
Low Back & High back L/S
& High back U/S

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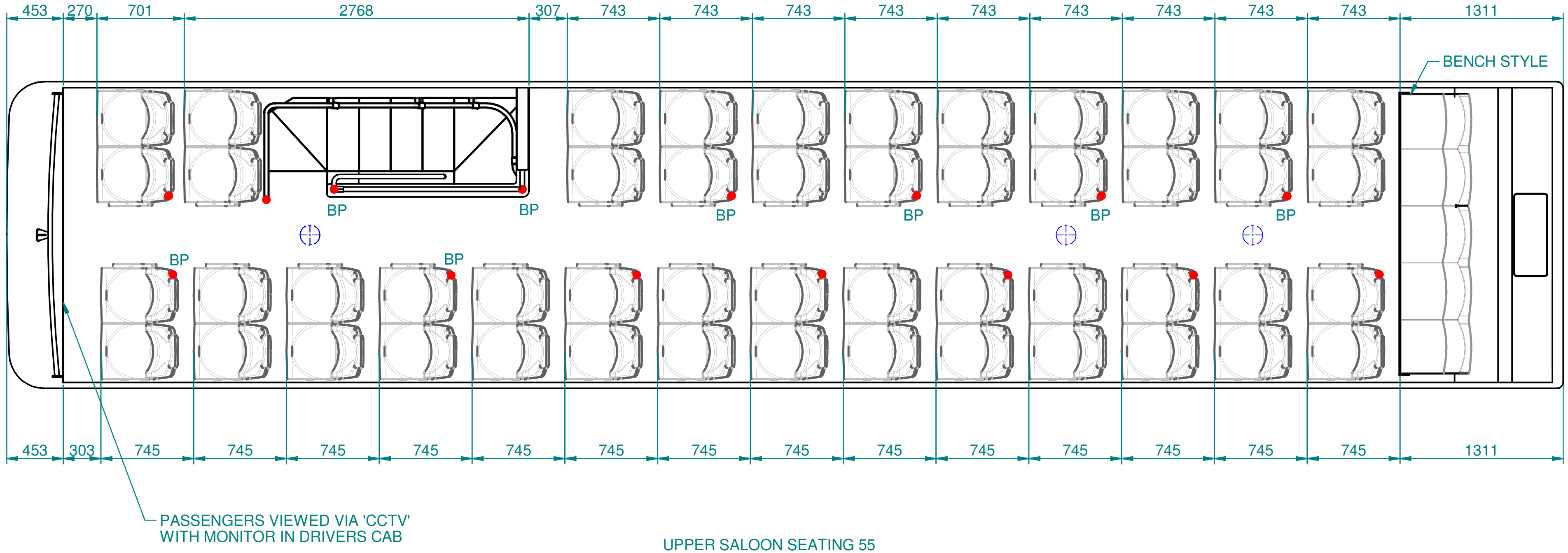
Drg No : WA4901



UPPER SALOON SEATING 55
PRACTICO LIGHTWEIGHT HIGHBACK DOUBLE 25
5 REAR SEAT BENCH STYLE 1

BP : BELL PUSH

ALEXANDER DENNIS

Rev	Date	Description	Name
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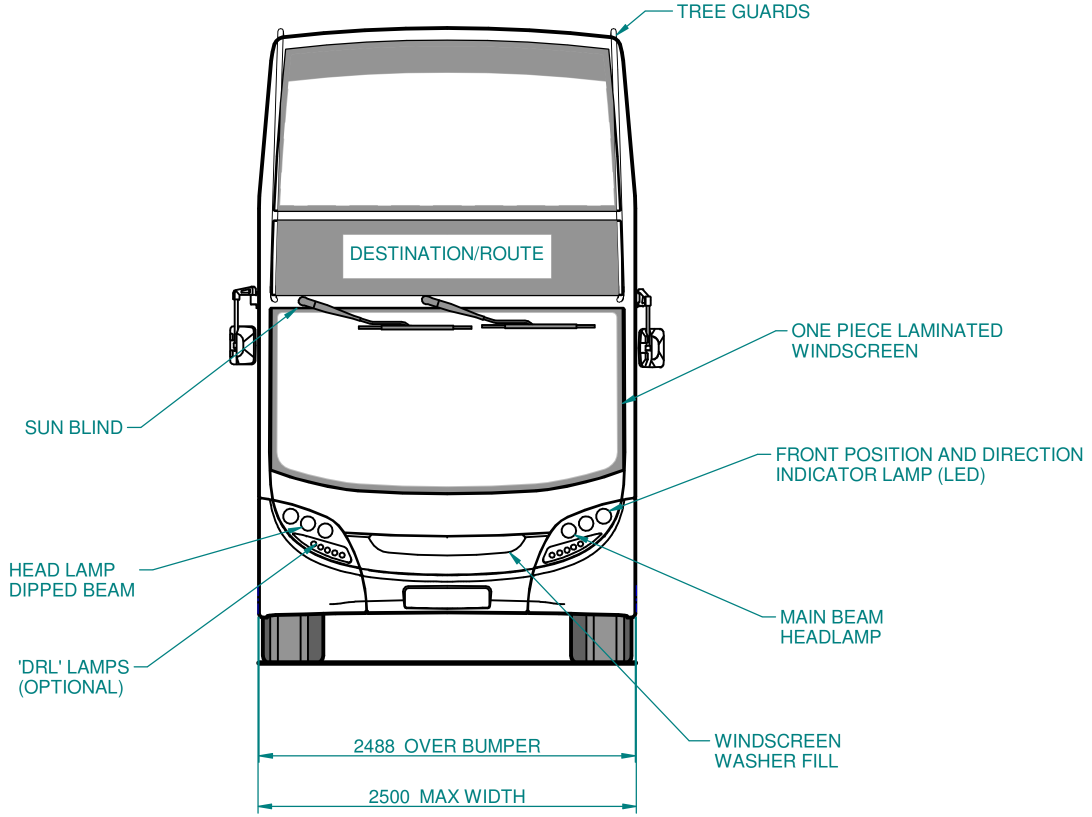
Page No : 4B
Drg No : WA4901

Form No EN-059



FRONT ELEVATION

ALEXANDER DENNIS

Form No EN-059

Rev	Date	Description	Name
01	18-04-13	RELEASE ISSUE	P.F.
02	21-05-13	WAS SINGLE EXIT DOOR - SEATING MODIFIED	P.F.
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REF : 'NEW ZEALAND'

Drawn : P.F.

Date : 11-04-13

Enq :

Seat Type :
Practico Lightweight
Low Back & High back L/S
& High back U/S

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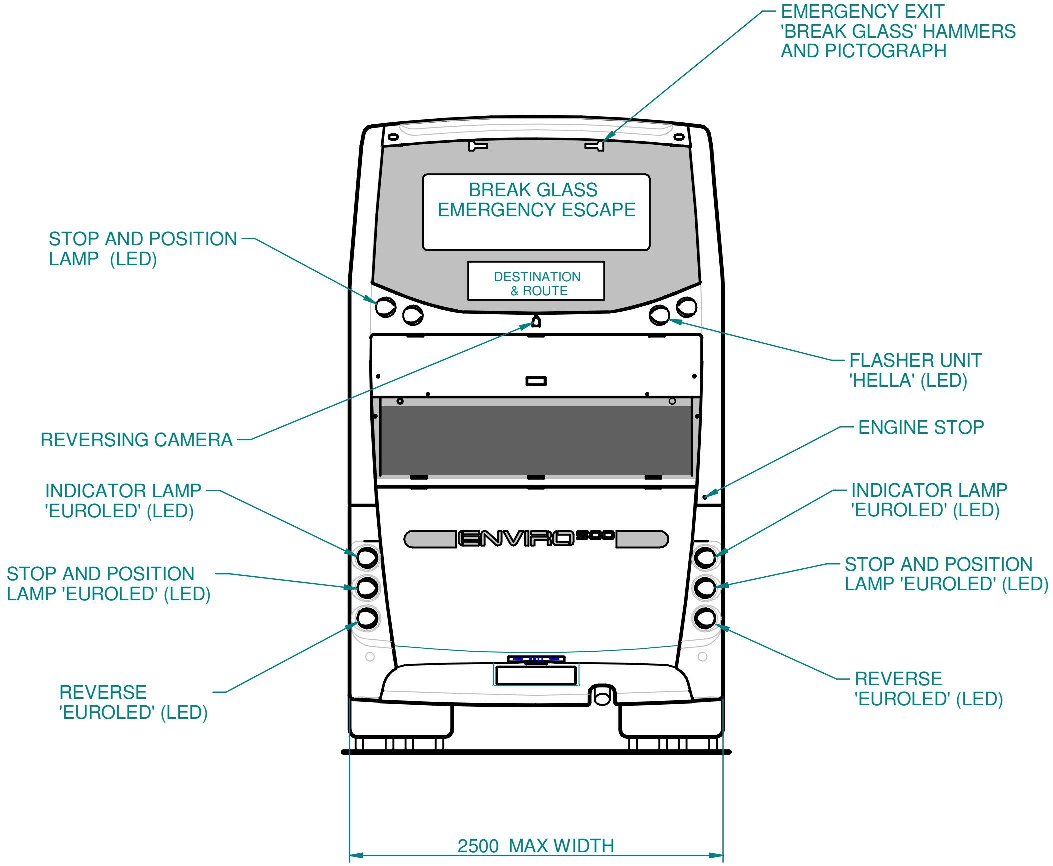

REAR ELEVATION

ALEXANDER DENNIS

Form No EN-059

Rev	Date	Description	Name
01	18-04-13	RELEASE ISSUE	P.F.
02	21-05-13	WAS SINGLE EXIT DOOR - SEATING MODIFIED	P.F.
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Date : 11-04-13

Enq :

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Low Back & High back L/S
& High back U/S

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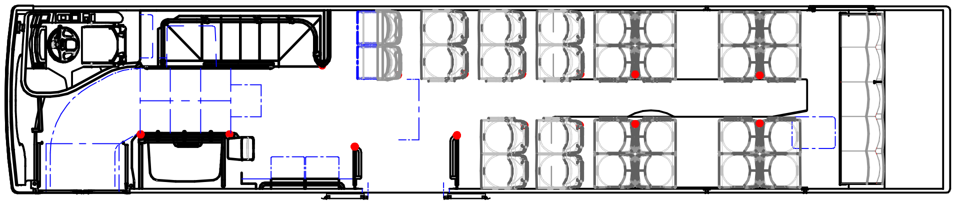
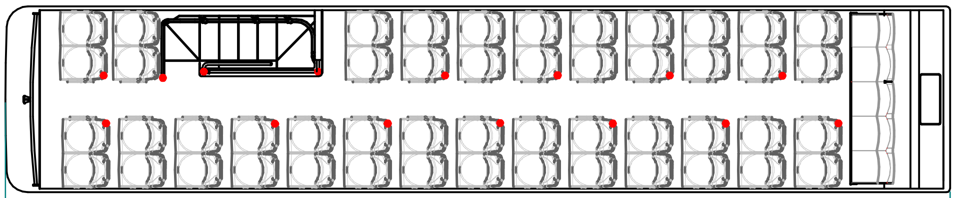
WA4901 - SHT 3B



LOWER SALOON : 35 SEATS
STANDING : 7

WA4901 - SHT 4B



UPPER SALOON : 55 SEATS

(12497)

LOWER SALOON : 35 SEATS
UPPER SALOON : 55 SEATS
STANDING : 7
TOTAL : 97

INCLUDING :
FRONT AXLE : 355/50R 22.5 TYRES
DRIVE AXLE : 275/70R 22.5 TYRES (55/45 SPLIT)
TAG AXLE : 355/50R 22.5 TYRES
ALUMINIUM FUEL TANK
LIGHT WEIGHT SEATS

NOTE : THIS ARRANGEMENT DOES CONFORM TO NEW ZEALAND AXLE / TYRE PERMITTED LOADS

DESCRIPTION	FRONT AXLE	REAR AXLE (55% / 45%)	TOTAL
ESTIMATED UNLADEN VEHICLE WEIGHT	4112 Kg	10851 Kg	14963 Kg
97 PASSENGERS + DRIVER @ 80 Kg	2794 Kg	5046 Kg	7840 Kg
ESTIMATED LADEN VEHICLE WEIGHT	6906 Kg	8743 Kg / 7154 Kg 15897 Kg	22803 Kg
VEHICLE MAX DESIGN WEIGHT	7000 Kg	10710 Kg / 7000 Kg 17710 Kg	24710 Kg
	-94 Kg	-1813 Kg	-1907 Kg
NZ MAX AXLE / TYRE PERMITTED LOADS	7000 Kg	8745 Kg / 7155 Kg 15900 Kg	22900 Kg
	-94 Kg	-3 Kg	-97 Kg

ALEXANDER DENNIS

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Drawn : P.F.
Date : 11-04-13
Enq :

Seat Type :
Practico Lightweight
Low Back & High back L/S
& High back U/S

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GA DRAWING NUMBER	WA6746-00 Max Standing		
CUSTOMER	NEW ZEALAND		
CHASSIS TYPE	ADL ELECTRIC - 472kWh		
BODY TYPE	MMC		
SEAT TYPE (S)	LAZZERINI PRATICO HIGH BACK LAZZERINI PRATICO LOW BACK		
SEATING CAPACITY L.S.	30	SEATED PASSENGERS	85
SEATING CAPACITY U.S.	55		
STANDEES	10	TOTAL	95
WHEELCHAIR (S) + PASSENGER (S)	0		
DRIVER	1	@ 80 Kg	
TYRES	Steer Axle : 355/50R 22.5 Drive Axle : 275/70R 22.5 Tag Axle : 355/50R 22.5		
PASSENGER MASS (Kg)	80		
WHEELBASE	6248		

VEHICLE WEIGHT DETAILS

ESTIMATED VEHICLE WEIGHTS :	STEER	DRIVE (55%)	TAG (45%)	TOTAL
VEHICLE WEIGHT (Kerbside)	4766 Kg	6470 Kg	5294 Kg	16531 Kg
WEIGHT DUE TO PASS & DRIVER	2377 Kg	2917 Kg	2386 Kg	7680 Kg
LUGGAGE (0 m³ @ 100 Kg / m³)	0 Kg	0 Kg	0 Kg	0 Kg
VEHICLE WEIGHT (Laden)	7143 Kg	9387 Kg	7680 Kg	24211 Kg
VEHICLE PLATED WEIGHT	7200 Kg	9777 Kg	8000 Kg	24977 Kg

P.S the passenger numbers are minimum and subject to increase when weighed when completed

Appendix C
CORRELATION BETWEEN DCP AND SUBGRADE CBR

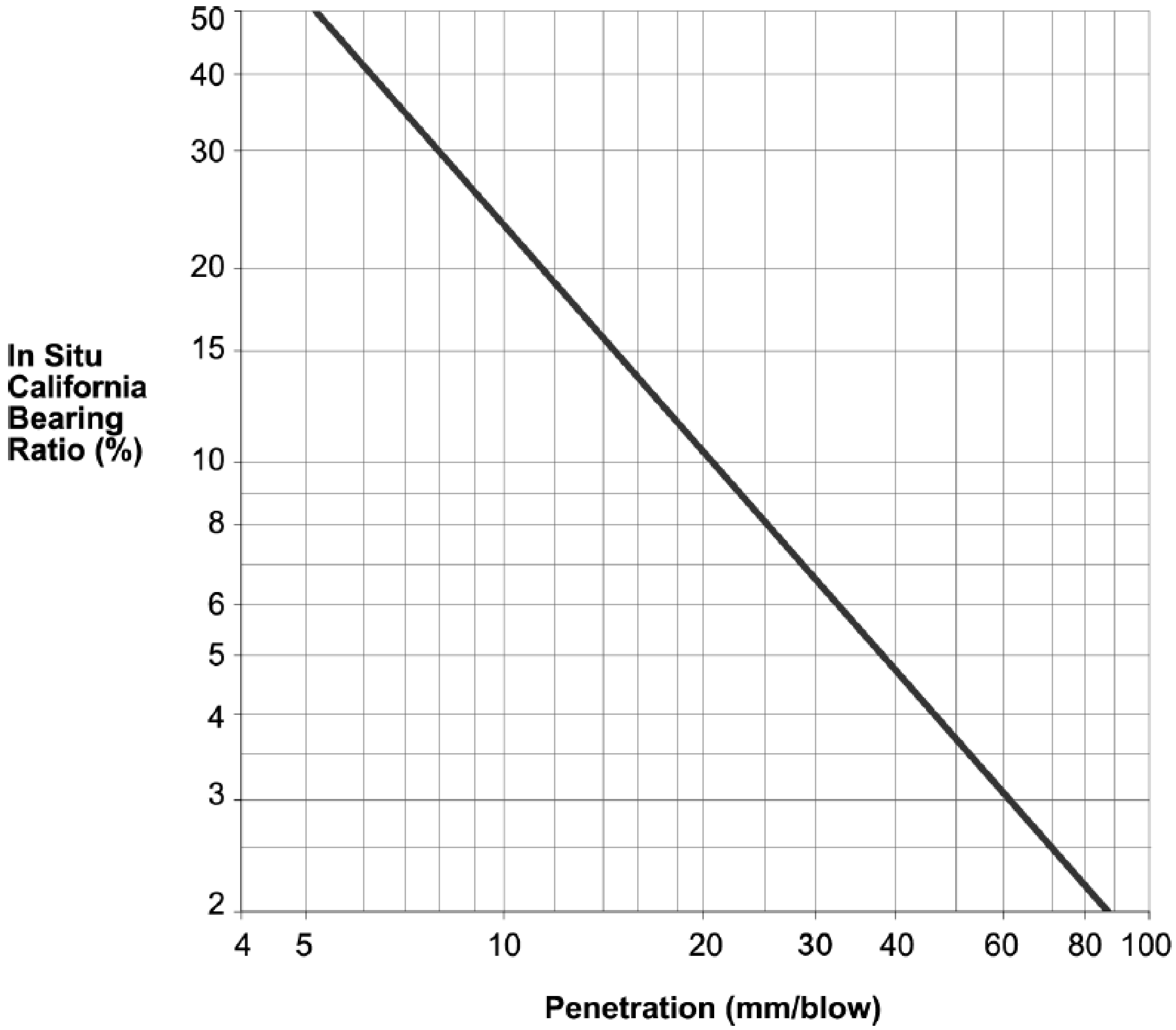
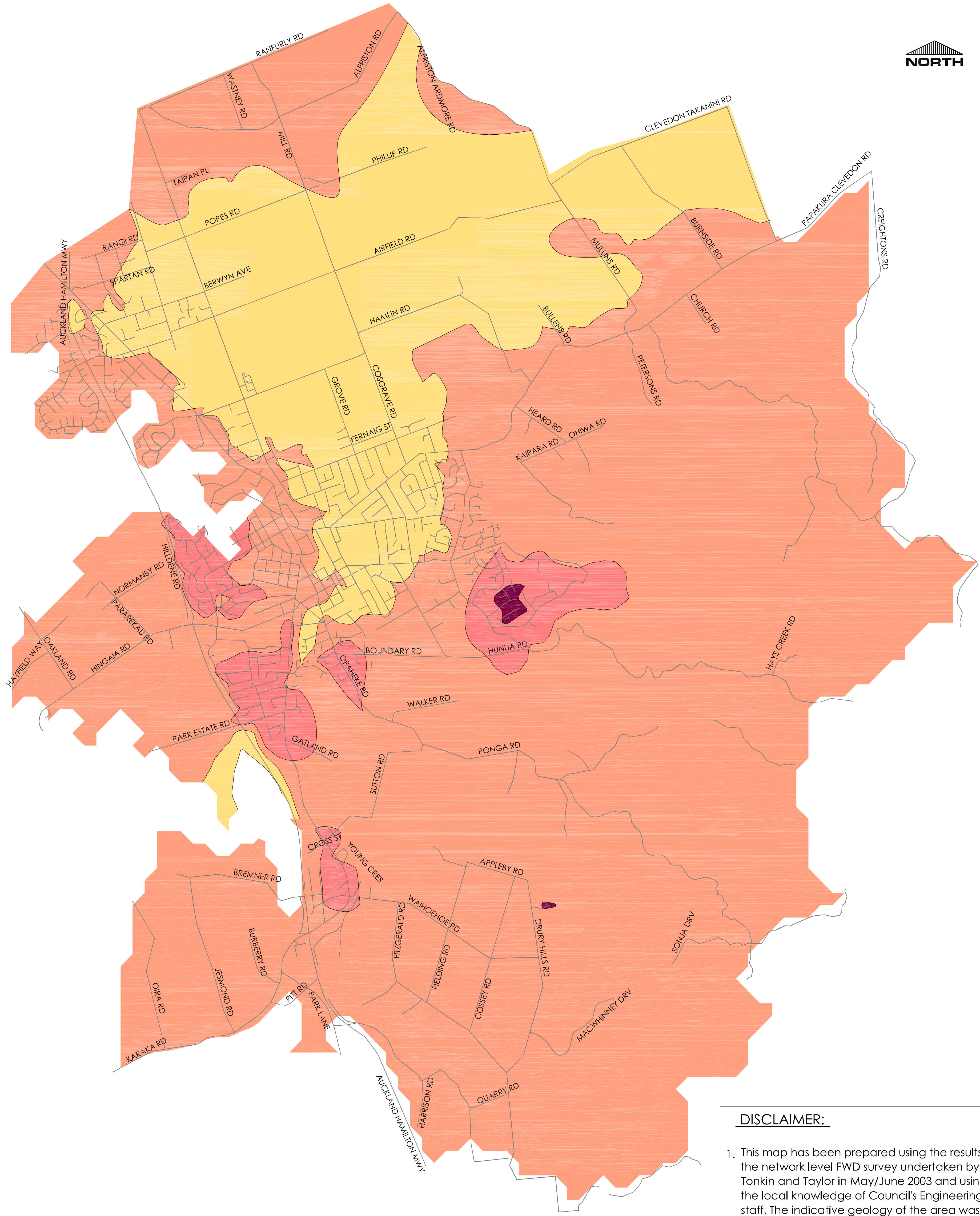


Figure 5.3: Correlation between dynamic cone penetration and CBR for fine-grained cohesive soils

Appendix D INDICATIVE SOIL MAP OF AUCKLAND



DISCLAIMER:

1. This map has been prepared using the results of the network level FWD survey undertaken by Tonkin and Taylor in May/June 2003 and using the local knowledge of Council's Engineering staff. The indicative geology of the area was obtained from the 1:250,000 series geology map for the auckland area.
2. This map is for information only and should not be used for design purposes. Site specific investigation should be used for the purposes of design.
3. Values for CBR have been determined from the back-analysis of FWD data. The values of CBR were then adjusted to reflect the local knowledge of Council's Engineering staff. Actual values in the field will vary substantially from location to location, and with season and depth.
4. There is very limited soils and geological information for Papakura District. Thus the soil types and inferred soil boundaries may vary significantly.

INDICATIVE SOIL TYPE

ALLUVIAL AND RESIDUAL DEPOSITS		SOILS OVERLYING SEDIMENTARY ROCKS AND VOLCANIC DEPOSITS	
PEAT	CLAY / SILT / SAND		

0 3 7 12+

CBR STRENGTH

SCALE 1 : 25,000 @ A1

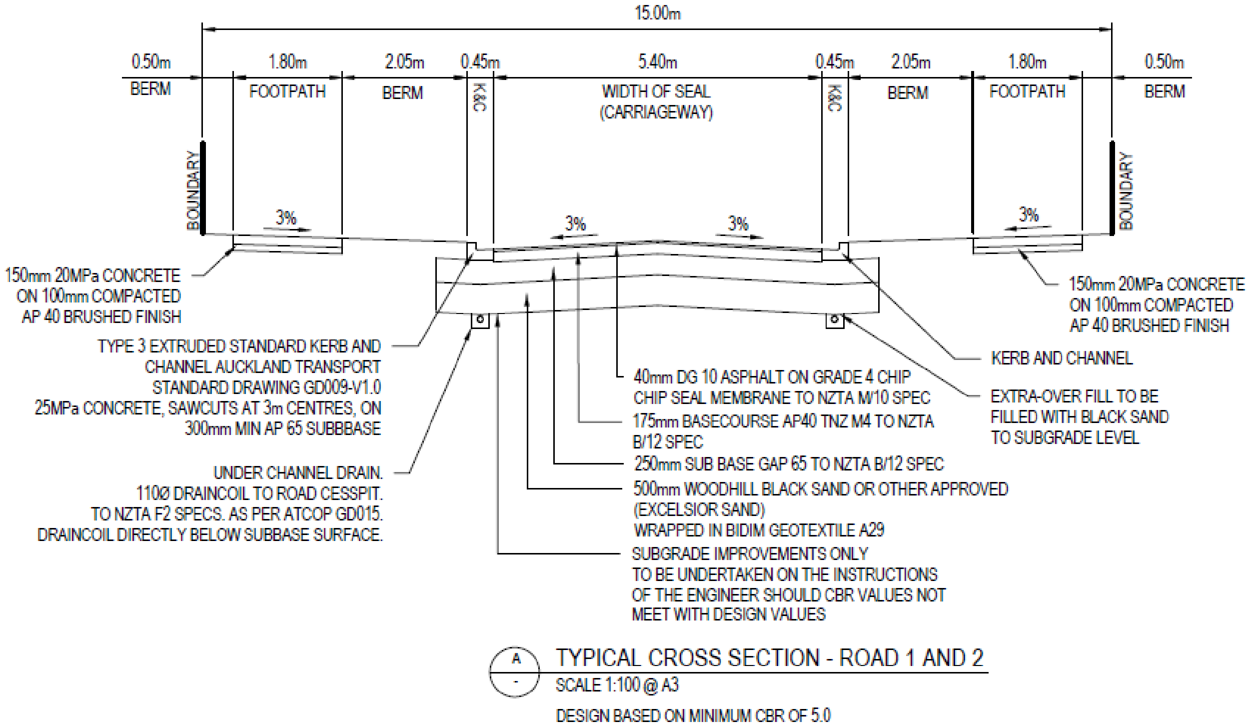
FOR: Papakura District Council

BY: BARTLEY CONSULTANTS
109 Great South Road, P.O. Box 26454, Epsom, Auckland, NZ.
Ph: (09) 529 0276
Fax: (09) 529 0297

project: PAPAKURA DISTRICT
CBR STRENGTH OF SOILS

issued for:	project number:	drawing number:	issue:
INFORMATION	0427-12	CBR01	0

Appendix E TYPICAL PAVEMENT – PAPAKURA DISTRICT COUNCIL



Sourced: ENG60384083 127 Grove Road (AT Approved Design – by Maven Associates)

Appendix F RESEALING & SEAL EXTENSION GUIDELINES

Reseal Guidelines

Asset Management and Systems

Auckland Transport
An Auckland Council Organisation

1. Guideline Definitions

Asphaltic Concrete (AC) is a dense, continuously graded mixture of coarse and fine aggregates, mineral filler and bitumen produced hot in a mixing plant. It is delivered, spread and compacted while hot. For the purposes of these guidelines, AC also includes slurry seals (due to its smooth finish) and other forms of asphalt.

AT - Auckland Transport

ATCOP - Auckland Transport Code of Practice

CCO – Council Controlled Organisation

Legal road has the same meaning as ***road*** in the Local Government Act 1974 (Section 315). In short, it covers the total area of land between road boundaries but is limited to the formed carriageway in these guidelines.

NZTA– New Zealand Transport Agency

Road Corridor has the same meaning as ***road*** in the Local Government Act 1974 (Section 315). In short, it covers the total area of land between road boundaries including:

- carriageway (formed road)
- footpath including kerb and channelling
- cycle ways, cycle paths
- walkways
- land that is legally designated as road but is not currently formed as carriageway or footpath

SOI – Statement of Intent

Terminology is used in this document to describe whether an aspect or statement is a requirement under law/mandatory or good practice:

- *Must* – indicates something that is mandatory or required by law
- *Should* – indicates a recommendation
- *May* – indicates something that is optional and may be considered for use.

2. Guideline Statement

The Mayor's vision outlines turning Auckland into the world's most liveable city by 2040. The Auckland Plan has identified that an efficient and integrated network of roads and public transport is vital to delivering this vision. As a Council Controlled Organisation (CCO), AT is responsible for delivering the region's transport services – from roads and footpaths to cycling, parking and public transport. Through the Statement of Intent (SOI) and to contribute to the achievement of priority areas and targets contained in the Auckland Plan, AT is required to prioritise and optimise investment across transport modes and related infrastructure.

AT has developed a set of guidelines to ensure that the transport services will be delivered on a consistent basis around the Auckland region. These guidelines identify the approach that AT will apply when managing the transport assets. The approach identified in the guidelines is cognizant with the Level of Service identified in the Integrated Transport Programme and Asset Management Plan (AMP).

The Auckland Transport Reseal Guidelines provide transparent criteria for determining the selection of the appropriate resurfacing material for resealing a road surface. With this in mind, it is appropriate to reseal most roads with chip seal unless circumstances exist where engineering best practice indicates that an asphaltic seal should be used.

Where practicable, reseal decisions are to take into account the whole-of-life cost of assets and consider an equitable allocation of resources.

3. Background

Following the transition from multiple local authorities to a single organisation, Auckland Transport is responsible for resealing the roads in the Auckland region. It is estimated that there is greater than 7,000 km of legal road around the region and Gulf Islands. These guidelines are required to provide the criteria for determining which resealing material to use when a road is due for resealing.

4. Purpose and Scope

Sealed roads provide greater skid-resistance and improved safety for road users. The purpose of these guidelines is to provide guidance on the principles and process for resealing decisions across the region.

The scope includes resealing of roads subject to high wear and tear, vehicle volumes and to roads subject to high pedestrian volumes. These guidelines provide the criteria for determining which seal material should be used and when.

The Reseal guidelines are aligned with the Street Amenities and Road Markings Guidelines. Technical specifications and engineering standards that related to the construction and maintenance of reseal or resurfacing works are provided in the Road Surfacing section of the Auckland Transport Code of Practice (ATCOP) and AT Road Network Asset Management Plan.

5. Guidelines

5.1 Selection of Seal Type

Chip seal surfacing must be used for resealing, except for roads which satisfy one or more of the criteria listed below, in which case asphaltic concrete surfacing must be used. The selection of the seal type in areas which fall within the Waitakere Ranges Heritage Area Act (2008) must comply with the provisions of this act.

Asphaltic concrete surfacing must be used for resealing roads:

a) Where the volume of traffic exceeds 10,000 vehicles per day, or
b) Subject to high wear and tear (such most cul-de-sac heads, roundabouts, sharp bends with severe flushing, stripping or skid resistance, aprons/main road intersections), or
c) In industrial/commercial areas where there is a high concentration of truck traffic, or
d) With short sections between two adjacent asphaltic concrete areas where the use of chip seal is uneconomic, or
e) Subject to high usage by pedestrians, such as town centres, hospitals, shopping centres and schools, or
f) Requiring special treatment due to the engineers discretion (such as steep gradients exceeding 15% or a cross-fall of >6%), or
g) Where intervention periods of greater than 20 years are required.

Resealing of CBD and other special amenity areas is subject to further assessment under the Street Amenities Guidelines, where the provisions for alternative paving and resealing materials must be considered.

All other road surfaces should be resealed with chip seal. In practice, this means that a low vehicle volume residential street that was previously sealed with asphaltic concrete (perhaps at the time of development) may be resealed with chip seal in accordance with the above criteria.

5.2 Seal Design

The actual seal material will, once the seal type has been selected under S5.1 above, be designed appropriately in accordance with the technical criteria stipulated in the ATCOP.

5.3 Reinstatement

Following the completion of the reseal works, the road markings must be reinstated in accordance with the Road Markings Guidelines unless otherwise directed by the AT Project Engineer.

5.4 Skid Resistance

Road surfaces must be constructed in accordance with the ATCOP and the standards therein to ensure sufficient skid resistance is provided.

6. Monitoring and Review

These guidelines shall be reviewed in 12 months and thereafter as part of the three year review cycle aligned to the Long Term Plan.

7. Related Guidelines

The detailed specifications for the provision of seal design in the road corridor are given in the ATCOP.

These guidelines also rely on the AT Street Amenities Guidelines, Seal Extension Guidelines and Road Marking Guidelines.

The Street Amenities Guidelines must be considered for the selection of the seal material in CBD or Town Centre areas, where replacement of like for like may be appropriate.

The assessment criteria used to determine whether to seal an unsealed road are discussed in the Seal Extension Guidelines.

8. Document Status

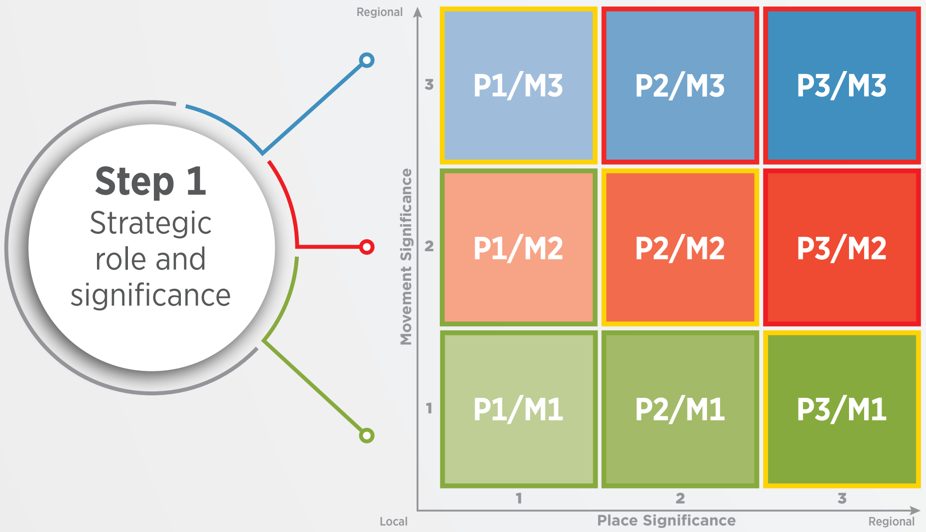
Owner (contact for updates, clarity etc.)	Siri Rangamuwa (Asset Management Planning Manager)	
Version no:	1.0 (Final)	
Issue date:	October 2013	
Review date:	October 2014	
Document ref no:	P-0001	Intranet Ref:

WRITTEN BY	Siri Rangamuwa **Asset Management Planning Manager**	01/11/13
ENDORSED BY	Tony McCartney **Group Manager Road Corridor**	6/4/13
APPROVED by	Andy Finch **Manager Strategic Asset Management and Systems**	21/2/14

Unsealed Road Improvement Framework decision making process

Part 1 - Need

Step 1 - Strategic Role and Significance

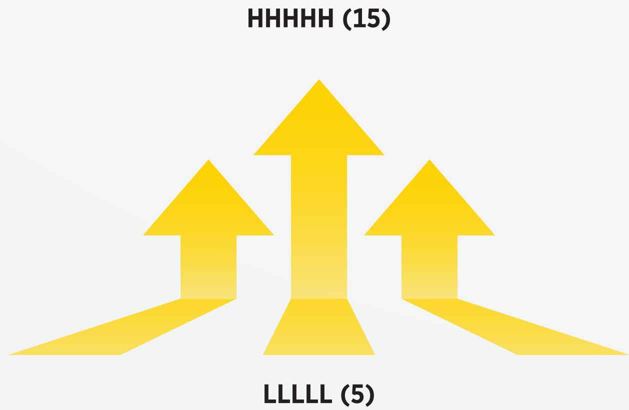


Traffic Volume

- **High (>200 vehicles per day)** Highest Strategic Significance
- **Medium (100 – 200 vehicles per day)** Medium Strategic Significance
- **Low (<100 vehicles per day)** Lowest Strategic Significance

Step 2 - Multi-criteria analysis

Step 3 - Results Ranking



Priority order based on five quantitative steps from 2 previous steps.

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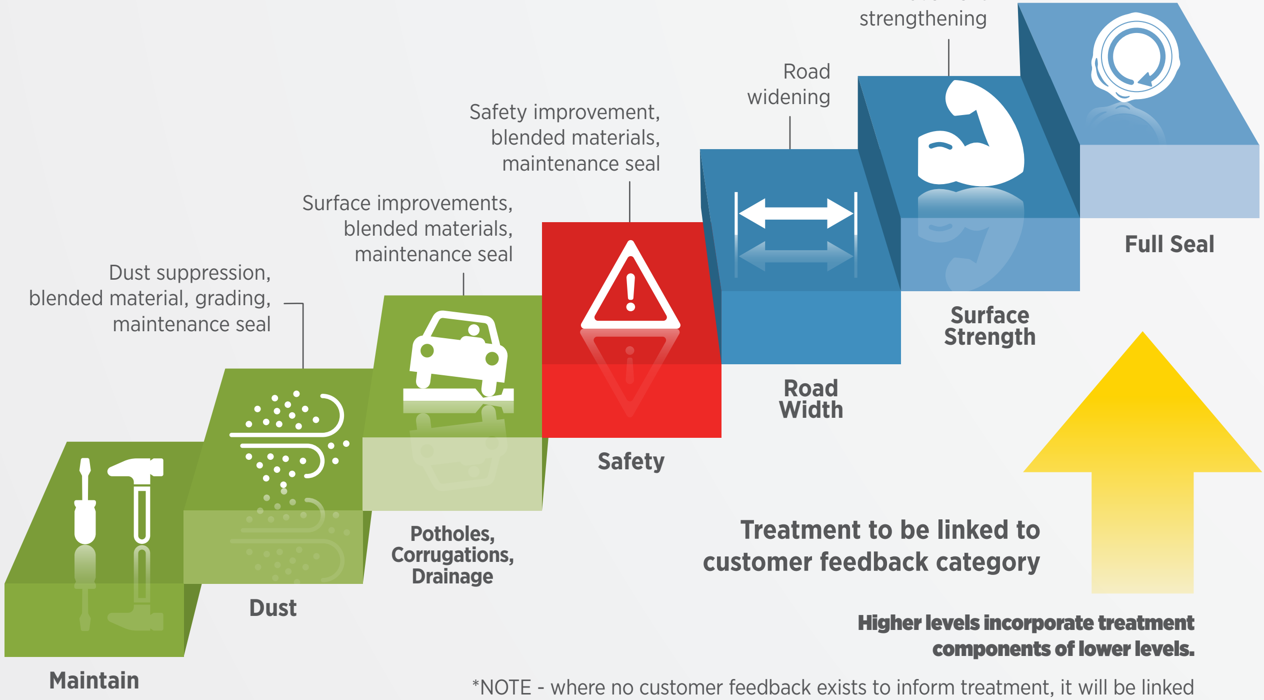
Unsealed Road Improvement Framework decision making process

Part 2 - Response

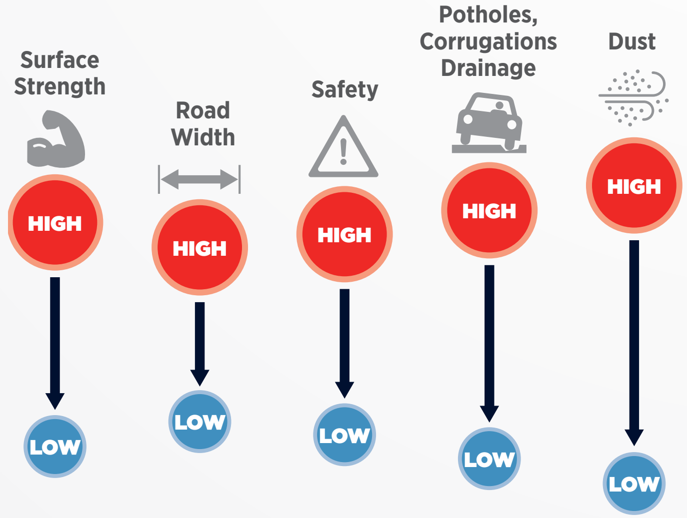
Step 4 - Treatment Escalation



Higher levels incorporate treatment components of lower levels.

NOTE - where no customer feedback exists to inform treatment, it will be linked to strategic role - with higher roles getting a higher treatment intervention.

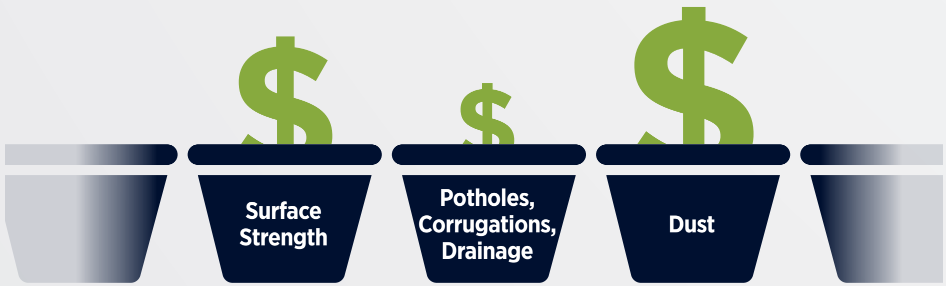
Step 5 - Treatment/Priority Matching



Priority of each road will be matched to the selected treatment option and prioritised from high to low.

Each treatment option will have its own prioritisation based on the scoring.

Step 6 - Budget Allocation



Funding

Dedicated budget to be set for each treatment option by Asset Management, given in accordance with priorities within each treatment and assigned on the basis of overall strategic objectives.

Where there are multiple issues, an integrated assessment will be completed.