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# Relations entre la composition, la structure et les propriétés thermo-mécaniques des matériaux bitumineux

François Olard

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N° d'ordre : 11-2013

# Mémoire d'Habilitation à Diriger des Recherches

présenté par

François OLARD

## Relations entre la composition, la structure et les propriétés thermo-mécaniques des matériaux bitumineux

HDR soutenue à l'Université Claude Bernard Lyon 1, membre de l'Université de Lyon,  
le 11 Mars 2013 devant la commission d'examen composée de :

M. Patrice HAMELIN, Université Claude Bernard Lyon 1	Président
M. François BUYLE-BODIN, Vice-Président de l'Université Lille 1	Rapporteur
M. Manfred PARTL, Royal Institute of Technology Stockholm & EMPA Zurich	Rapporteur
M. Christophe PETIT, Université de Limoges	Rapporteur
M. Ali LIMAM, Institut National des Sciences Appliquées de Lyon	Examineur
M. Daniel PERRATON, Ecole de Technologie Supérieure de Montréal	Examineur
M. Philippe TAMAGNY, Institut Français des Sciences et Technologies des Transports, de l'Aménagement et des Réseaux	Examineur
M. Hervé DI BENEDETTO, Ecole Nationale des Travaux Publics de l'Etat	Directeur HDR



*A ma femme adorée et à mes cinq enfants chéris*

*A ma mère*

*A la mémoire de mon père et de mes grands-parents*

*A tous ceux qui, de près ou de loin, ont contribué à ce travail*



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# Chapitre I. Curriculum Vitæ

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Né à Dreux en 1977  
 Français, marié  
 5 enfants

## 1.1 Formation et diplômes

- 2000-2003: **Thèse de Doctorat en Génie Civil** de l'Ecole Doctorale MEGA de l'Université de Lyon (Centrale Lyon-INSA Lyon-Ecole Nationale des TPE-UCBL).

Thèse: « *Comportement thermomécanique des enrobés bitumineux à basses températures. Relations entre les propriétés du liant et de l'enrobé* ».

Jury: Richard KASTNER	Président
Antonio GOMES CORREIA	Rapporteur
Denys BREYSSE	Rapporteur
Bernard HERITIER	Examineur
Didier LESUEUR	Examineur
Hervé DI BENEDETTO	Directeur

- 1999-2000: **Diplôme d'Etudes Approfondies (DEA) en Génie Civil** de l'Ecole Doctorale MEGA de l'Université de Lyon (Centrale Lyon-INSA Lyon-Ecole Nationale des TPE-UCBL), Mention Très Bien.
- 1997-2000: **Diplôme d'ingénieur** civil de l'ENTPE (Ecole Nationale des Travaux Publics de l'Etat), Lyon, Mention Très Bien.
- 1995-1997: **Classes Préparatoires** (Mathématiques Supérieures et Spéciales, option PSI\*), Lycée Stanislas (Paris).

## 1.2 Langues

- Anglais courant, notions de Japonais et d'Espagnol.

## 1.3 Activités professionnelles

- 2013-présent: **Directeur de la Recherche et de l'Innovation** EIFFAGE TRAVAUX PUBLICS
- 2003-2012: **Chef de Projets R&D** EIFFAGE TRAVAUX PUBLICS:
  - Coordination scientifique et administrative du projet RECYROUTE (projet ANR 2008-2012, appel PGCU 2007, 2 400 000€)
  - Portfolio Propriété Intellectuelle et industrielle (matériaux, procédés, équipements)

- Collaborations avec les services Technique et Matériel d'EIFFAGE TRAVAUX PUBLICS (enrobés tièdes, recyclage à fort taux, béton compacté fibré)
- Collaboration avec le service Technique EIFFAGE CONSTRUCTION METALLIQUE (ponts à dalle métallique orthotrope, projet ANR ORTHOPLUS 2007-2011)
- Collaborations avec laboratoires académiques (ENTPE (viscoélasticité et fatigue des enrobés), IFSTTAR (évaluations environnementales), Sétra (innovation routière) ou laboratoires privés (inter-comparaison))
- Supervision de 4 thèses de Doctorat et de 23 stages étudiants
- 2010-présent: **Professeur Associé à l'Ecole de Technologie Supérieure (ETS)**, Université du Québec à Montréal (UQAM):
  - Collaboration avec le Prof. Daniel Perraton (viscoélasticité, optimisation granulaire) et le Prof. Alan Carter (performances des enrobés tièdes)
  - Enseignement du cours « Routes » (18h)
- 2004-présent: **Membre de l'Editorial Board** de l'International Journal of Road Materials & Pavement Design (IJRMPD, Taylor & Francis), ISI (Institute for Scientific Information).
- 2005-présent: **Membre du Comité Technique LEACO** (Low Energy Asphalt COmpany).
- 2004-présent: **Reviewer** des revues **IJRMPD** (Taylor & Francis), **Materials & Structures** (Rilem), **Materials in Civil Engineering & Transportation Engineering** (Am. Society of Civil Engineers), référencées ISI.
- 2008-présent: **Membre du Comité Scientifique** des conférences bisannuelles de l'EATA (European Asphalt Technologists Association).
- 2012-présent: **Membre du Conseil Stratégique de l'Institut des Procédés Appliqués aux Matériaux** (IPAM) de l'Université de Limoges.
- 2012-présent: **Intervenant et coordinateur** avec Y. Brosseau (IFSTTAR) de la formation « Les enrobés tièdes: formulations, caractéristiques, fabrication et mise en œuvre » à l'**ENPC Formation Continue** (2 jours).
- 2012-présent: **Intervenant** du cours « Les matériaux routiers » à l'**ENTPE** (cours N°7 (sur 9) de 3 heures traitant de la fabrication, de la mise en œuvre et du contrôle des matériaux).
- 2012-présent: **Intervenant** du cours « Initiation au BTP » à l'**Ecole des Mines de Saint-Etienne** (EMSE) (cours de 3 heures traitant de la construction routière).

#### 1.4 Activités passées

- 2008-09: Enseignant à l'ENTPE (cours de 6 heures traitant des matériaux bitumineux).
- 2002-04: Enseignant à l'ENTPE (cours de 57 heures de Mécanique des Milieux Continus).
- 2002-03: Travaux Pratiques à l'ENTPE de RDM (10 heures) et de Routes (10 heures).
- 2000-03: Thèse en convention CIFRE au sein du laboratoire central EUROVIA (groupe VINCI) sous la supervision du Prof. Hervé Di Benedetto (ENTPE). Titre: Comportement thermo-mécanique des enrobés bitumineux à basses températures. Relations entre les propriétés du liant et de l'enrobé.

## 1.5 Distinctions et Prix

- 2003, 2007 & 2010: Road Materials & Pavement Design awards (Best Paper award).
- 2004: Prix du Groupe Français de Rhéologie (Prix de la meilleure thèse de Doctorat).
- 2004: 1<sup>er</sup> Prix de la Fondation Excellence SMA (groupe SMABTP).
- 2007: Prix mondial de la Route (catégorie Développement Durable) de l'Association mondiale de la Route (AIPCR): invention des enrobés à basse température EBT<sup>®</sup>
- 2007: Trophée Innovation EIFFAGE: invention des enrobés à basse température EBT<sup>®</sup>
- 2008: Prix Développement Durable & Prix Charles Parey du Comité Français AIPCR (World Road Association).
- 2011: 2<sup>e</sup> Prix de l'innovation du Comité Français AIPCR (World Road Association).
- 2011: 2<sup>e</sup> Prix du Trophée Eugène FREYSSINET: invention des enrobés bitumineux à hautes performances GB5<sup>®</sup> (optimisation granulaire & bitumes polymères)

## 1.6 Résumé de la production scientifique et index de citation

- Supervision de 23 stages étudiants (diplômes Ingénieurs et Mastères).
- Supervision de 4 doctorants (Génie Civil).
- Participation aux Jurys de 4 thèses de Doctorat en Génie Civil.
- 2 Chapitres d'ouvrages scientifiques.
- 23 publications au sein de revues internationales à comité de lecture.
- 3 publications au sein de revues françaises à comité de lecture.
- 54 publications et présentations en conférences internationales.
- 30 publications de vulgarisation dans la littérature technique & professionnelle.
- 3 brevets internationaux.
- 16 référencements WoS (Web of Science), la revue RMPD n'étant référencée SCI (Science Citation Index) qu'à partir de 2005. Plus largement, 81 référencements selon Harzing.com (Publish or Perish software 3.7).
- H-number<sup>1</sup> : 4 selon WoS (9 selon Harzing) et G-number<sup>2</sup> : 5 selon WoS (13 selon Harzing).
- 2,27 citations par publication (3,31 selon Harzing).
- 13 citations pour l'article suivant selon WoS: *Three-dimensional thermo-viscoplastic behavior of bituminous materials: the DBN model*, avec H. Di Benedetto, M. Neifar, C. Sauzéat.

29 citations pour l'article suivant selon Harzing: *Linear viscoelastic behavior of bituminous materials: from binders to mixes*, avec H. Di Benedetto, C. Sauzéat, B. Delaporte.

<sup>1</sup> C'est le plus grand entier  $h$  tel que les  $h$  publications les plus citées le sont au moins  $h$  fois chacune.

<sup>2</sup> C'est le plus grand entier  $g$  tel que les  $g$  publications les plus citées recueillent ensemble au moins  $g^2$  citations. Si les citations sont à peu près uniformes, le  $g$ -number est égal au  $h$ -number. Le  $g$ -number favorise la présence d'articles-phare particulièrement cités et a été proposé comme une amélioration du  $h$ -number qui évalue plus finement l'impact des scientifiques.



# Chapitre II. Activités d'enseignement, de recherche et animation d'équipe

## 2.1 Activités d'enseignement

Enseigner est aujourd'hui primordial pour moi et fait partie intégrante de ma démarche et de ma mission de recherche. Pourtant, ma passion pour l'enseignement en Génie Civil ne remonte ni à mon environnement familial, ni à un souvenir particulier en tant qu'élève ingénieur. Elle naît à l'occasion de mon Master Recherche (anciennement DEA (diplôme d'études approfondies)) pendant lequel je développe une attitude à la fois critique et constructive sur les cours qui me sont donnés. Le cours de Thermodynamique des Milieux Continus du Prof. F. Sidorov à l'Ecole Centrale de Lyon (ECL) m'amène en particulier à relire avec application le cours de Mécanique des Milieux Continus que le Prof. P. Royis m'avait enseigné deux ans auparavant, en première année à l'Ecole Nationale des Travaux Publics de l'Etat (ENTPE). Par ailleurs, le cours de Rhéologie qu'enseigne le Prof. H. Di Benedetto en dernière année à l'ENTPE, très orienté recherche, me conduit à m'interroger sur les méthodes d'enseignement, sensiblement différentes, retenues par les uns et par les autres. L'ensemble de ces cours, très liés et complémentaires, constitue le véritable déclic.

Apprécier le contenu d'un cours et la façon dont celui-ci est enseigné est une chose. Enseigner soi-même devant une salle de classe ou un amphithéâtre en est une autre ! P. Royis me propose, courant 2002, d'enseigner le cours de Mécanique des Milieux Continus à une des classes de 30 élèves de la jeune promotion qui intégrera l'ENTPE en septembre. Jeune doctorant (fin de deuxième année de doctorat), j'accepte avec beaucoup de plaisir et d'engouement. Je passe cependant tout l'été à préparer au mieux les 57 heures de cours et de travaux pratiques en puisant dans mes propres cours et dans ceux empruntés alors à D. Branque et H. Geoffroy, du Département Génie Civil et Bâtiment (DGCB) de l'ENTPE.

Cette première expérience d'enseignement dure deux années scolaires, je dois y mettre fin en effet en 2004, mes nombreuses absences par demi-journées étant peu compatibles avec mes nouvelles responsabilités d'ingénieur recherche au sein d'EIFFAGE TRAVAUX PUBLICS (anciennement APPIA) depuis 2003. Ces deux années d'enseignement, très denses entre septembre et décembre, sont néanmoins une formidable expérience, et ce à double titre :

- d'une part, ce cours de Mécanique des Milieux Continus en première année à l'ENTPE est caractérisé par son approche fondamentale et sa forte teneur mathématique. Le cours ouvre –par le biais d'exercices et d'applications– des perspectives directes sur les cours qui suivent au sein de l'école (géotechnique, résistance des matériaux, rhéologie, etc.).
- d'autre part, l'hétérogénéité du groupe d'élèves nécessite une approche pédagogique des plus intéressantes. En effet, même si la majeure partie des élèves est issue des Classes Préparatoires (filiales MP, PC, PSI principalement), certains élèves sont des EX-AT, ex-assistants techniques du ministère en charge de l'équipement, au sortir d'une année de cycle probatoire (remise à niveau à l'ENTPE). Ainsi, pendant le cours magistral, je privilégie la transcription progressive au tableau, de manière à pouvoir m'adapter en temps réel aux difficultés rencontrées par les uns ou les autres. Bénéficiant enfin du fait que la classe soit constituée d'une trentaine d'élèves, je privilégie autant que faire se peut la participation active et la prise de parole des étudiants, leur proposant régulièrement d'échanger en reformulant une partie du cours ou en résolvant un petit exercice.

En parallèle à cet enseignement de la MMC, j'ai également pu encadrer quelques séances de travaux pratiques à l'ENTPE entre 2002 et 2003, d'une part de Résistance des matériaux (RDM) (10h) et d'autre part de Routes (10h).

En 2008 et 2009, au sein du cours de première année Matériaux du Génie Civil (MGC) de l'ENTPE, j'ai présenté à deux classes (2x3h) les « Liants hydrocarbonés et leurs applications routières ».

Actuellement, mes principales activités d'enseignement sont les suivantes :

- Depuis 2011, à l'Ecole de technologie supérieure (ETS) de l'Université du Québec à Montréal (UQAM): cours de Maîtrise MGC 922, « Matériaux Bitumineux : Formulation, Fabrication et Mise en Place » (36h), co-supervisé avec le Prof. D. Perraton.

Ce cours a pour but de permettre à l'étudiant de maîtriser les notions fondamentales de formulation, caractérisation, fabrication et mise en place des matériaux bitumineux. Au terme de ce cours, l'étudiant est en mesure:

- d'appréhender le comportement mécanique d'une structure de chaussée et son évolution potentielle vis-à-vis des effets combinés du trafic et du climat.
- de comprendre la méthode de formulation et de caractérisation en laboratoire des matériaux bitumineux (aptitude au compactage, résistance à l'eau, résistance à l'orniérage (accumulation de déformations permanentes), module de rigidité complexe ou sécant, résistance en fatigue).
- d'utiliser des modèles analogiques (modèles Kelvin-Voigt ou Maxwell généralisés, Huet-Sayegh, 2S2P1D) pour traduire le comportement viscoélastique linéaire en petites déformations des matériaux bitumineux.
- de comprendre les méthodes rationnelles de dimensionnement de chaussées, françaises et québécoises, en intégrant les caractéristiques de laboratoire et le trafic.
- de différencier le mode de fabrication des enrobés à chaud, des enrobés dits tièdes ou semi-tièdes, des enrobés à froid, fabriqués soit en centrales d'enrobage (fixes ou mobiles), soit *in-situ*.
- de comprendre les méthodes de recyclage des enrobés, avec incorporation à froid ou à chaud des recyclés, en centrale ou *in-situ*.
- de comprendre les impacts environnementaux et économiques associés à la fabrication et à la mise en œuvre des liants et des enrobés bitumineux (avantages et désavantages associés à chaque technique de fabrication et de mise en œuvre).

Ce cours a pour particularité d'être donné à raison de trois fins de semaine intensives (vendredi et samedi). En outre, il donne lieu à un projet préparé en binôme, déterminé à partir d'une liste imposée de sujets. Hormis un document de synthèse d'une vingtaine de pages à produire, le binôme présente son travail à ses camarades pendant dix minutes, tandis que cinq minutes d'échange avec la classe permettent de transmettre à l'ensemble du groupe une partie des connaissances acquises dans l'exercice de ce travail par le binôme.

- Depuis 2012, dans le cadre de la formation continue de l'ENPC (Ecole nationale des ponts et chaussées), je supervise avec Y. Brosseau (IFSTTAR) la session de formation de deux jours intitulée « Enrobés tièdes: formulations, caractéristiques, fabrication et mise en œuvre ».

Sont abordées pendant ce cours les thématiques suivantes:

- Etat des lieux des différents procédés et techniques d'enrobés tièdes.
- Incidences sur la fabrication et la mise en œuvre.
- Avantages environnementaux, économiques et techniques
- Essais de caractérisation.
- Témoignage d'un maître d'ouvrage.
- Pistes de normalisation.

Dans le cadre de cette formation continue ENPC, les stagiaires sont des responsables de services techniques départementaux, de services techniques de villes ou des services du ministère en charge de la construction et de l'entretien des chaussées (Ministère de l'Ecologie, du Développement durable et de l'Energie, MEDDE), responsables de

laboratoires, de bureaux d'études, d'entreprises routières et d'entreprises productrices de matériaux d'enrobés. Du point de vue pédagogique, les supports utilisés sont ici principalement des présentations de type PowerPoint, tout en privilégiant les échanges avec les stagiaires autant que possible.

- Depuis 2012, j'interviens dans le cours Matériaux Routiers (MR) de l'ENTPE, coordonné par Y. Brosseaud (IFSTTAR). Je donne plus particulièrement le cours N°7 « Fabrication, mise en œuvre et contrôle des matériaux » (3h).
- Depuis 2012, j'interviens dans le cours « Initiation au BTP » à l'Ecole des Mines de Saint-Etienne (EMSE). Je donne plus particulièrement le cours consacré à la construction routière (3h).
- A partir de 2013, projet de formation continue ENPC « Recyclage des enrobés », sous forme d'une session de deux jours coordonnée avec Y. Brosseaud (IFSTTAR).

Mes principales activités d'enseignement passées ou en cours peuvent donc se résumer comme suit :

<b>Discipline Heures</b>	<b>Années</b>	<b>Niveau</b>	<b>Etablissement</b>
Cours et TP MMC 57h	2002-2004	Bac+3	ENTPE
TP RDM 10h	2002-2003	Bac+3	ENTPE
TP Routes 10h	2002-2003	Bac+3	ENTPE
Cours MGC 6h	2008-2009	Bac+3	ENTPE
Cours MGC 18h	2011-présent	Bac+5	ETS Montréal
Cours Enrobés tièdes 2 jours	2012-présent	Techniciens et ingénieurs en fonction	Formation Continue ENPC
Cours MR 3h	2012-présent	Bac+3	ENTPE
Cours Initiation au BTP 3h	2012-présent	Bac+4	Mines Saint-Etienne
Cours Recyclage 2 jours	2013	Techniciens et ingénieurs en fonction	Formation Continue ENPC



## 2.2 Activités de recherche et d'animation d'équipe

### **2.2.1 Encadrement d'élèves ingénieurs, de mastères et de doctorants**

#### Supervision de 23 stages étudiants (diplômes Ingénieurs et Mastères)

- **M. Chalamet** (2004), « Formulation d'Enrobés Tièdes (130°C) en laboratoire », Hautes Etudes d'Ingénieur (HEI) de Lille.
- **G. Siewe** (2004), « Etude du comportement des revêtements de dalle orthotrope. Cas du viaduc de Millau », Ecole Nationale des Ponts et Chaussées (ENPC). Co-encadré avec B. Héritier (APPIA).
- **E. Beduneau** (2005), « Etude des Matériaux Traités à la Mousse de Bitume: Influence de la température et de la teneur en eau sur les propriétés mécaniques », Institut universitaire professionnalisé (IUP) de Poitiers. Co-encadré avec A. Beghin et B. Cazacliu (IFSTTAR Nantes).
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- **D. Pellerin** (2010-2011), « Comportement viscoélastique linéaire et à la rupture des graves-bitume GB5<sup>®</sup> », Ecole de Technologie Supérieure (ETS), Université du Québec.
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- **S. Lecomte** (2011), « Formulation d'une micro-GB5<sup>®</sup> et développement d'un essai de fatigue court sur enrobés », Université de Poitiers.
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- **N. Levers** (2012), « Enrobage séquentiel des enrobés », Université de Technologie de Troyes (UTT).

Supervision de 4 doctorants en Génie Civil (Ecole doctorale MEGA, Université de Lyon)

- **S. Pouget**, « Influence des propriétés élastiques ou viscoélastiques des revêtements sur le comportement des ponts à dalle orthotrope ». Thèse, en convention CIFRE (N° 2008/1199), soutenue le 15 mars 2011 devant l'ENTPE, Université de Lyon.

Jury:	LIMAM Ali	Président
	PARTL Manfred N.	Rapporteur
	TORRENTI Jean-Michel	Rapporteur
	DI BENEDETTO Hervé	Directeur
	OLARD François	Tuteur
	SAUZEAT Cédric	Tuteur
	RAOUL Joël	Examineur
	DUBUJET Philippe	Examineur

Co-encadrement :

Prof. H. Di Benedetto (20%)  
 Dr. C. Sauzeat (40%)  
 Dr. F. Olard (40%)

Publications : les travaux de recherche de S. Pouget ont donné lieu à 6 publications scientifiques de rang A [17][18][22][23][24][25], 7 communications en congrès [45][62][63][72][73][74][75] et 2 articles de vulgarisation [106][114].

Suite à sa thèse, S. Pouget a intégré le Centre d'Etudes et de Recherches EIFFAGE Travaux Publics à Corbas (69).

- **K. Bilodeau**, « Evaluation mécanique des matériaux composites incluant des fraisats bitumineux traités au liant hydraulique ». Thèse, en convention CIFRE (N° 2009/0284), soutenue le 30 novembre 2012 devant l'ENTPE, Université de Lyon.

Jury:	LIMAM Ali	Président
	PERRATON Daniel	Rapporteur
	NEIFAR Monder	Rapporteur
	DI BENEDETTO Hervé	Directeur
	OLARD François	Tuteur
	SAUZEAT Cédric	Tuteur
	BALAY Jean-Maurice	Examineur

Co-encadrement :

Prof. H. Di Benedetto (20%)  
 Dr. C. Sauzeat (40%)  
 Dr. F. Olard (40%)

Publications : les travaux de recherche de K. Bilodeau ont donné lieu à 1 projet en cours de publication scientifique de rang A (Materials & Structures), 5 communications en congrès [54][64][67][68][76] et 1 article de vulgarisation [108].

Suite à sa thèse soutenue le 30 novembre 2012, K. Bilodeau est en cours de discussion avec deux entreprises québécoises pour en devenir responsable technique.

- **S. Mangiafico**, « Propriétés viscoélastiques linéaires et à la fatigue des liants et des enrobés bitumineux ».  
Soutenance de thèse prévue fin 2013 devant l'ENTPE, Université de Lyon.

Co-encadrement :

Prof. H. Di Benedetto (20%)  
Dr. C. Sauzeat (30%)  
Dr. F. Olard (30%)  
Dr. S. Pouget (20%)

Publications : les travaux de recherche de S. Mangiafico ont donné lieu à 2 projets en cours de publications scientifiques de rang A [26][27] et 3 présentations en congrès [79][80][81].

- **P. Gaborit**, « Comportement structural des chaussées bitumineuses épaisses du réseau autoroutier AREA ».  
Soutenance de thèse prévue fin 2014 devant l'ENTPE, Université de Lyon.

Co-encadrement :

Prof. H. Di Benedetto (20%)  
Dr. C. Sauzeat (25%)  
Dr. S. Pouget (25%)  
Dr. F. Olard (20%)  
S. Quivet (10%)

Publications : les travaux de recherche de P. Gaborit ont d'ores et déjà donné lieu à 2 présentations en congrès [77][78].

## 2.2.2 Animation de la recherche

### Pilotage de projets ANR

- **Projet ORTHOPLUS** (avril 2007-avril 2011), « Ingénierie avancée des dalles orthotropes et de leur revêtement pour une optimisation globale de leur cycle de vie ».

Thème : Le projet ORTHOPLUS a été lauréat de l'appel à projets de recherche du Réseau Génie Civil et Urbain (RGCU) lancé par l'Agence Nationale de Recherche (ANR) en 2006. L'objectif est de développer une nouvelle conception des dalles orthotropes et de leur revêtement, des études théoriques aux applications techniques.

Partenaires : 2 entreprises privées (EIFFAGE Travaux Publics, EIFFAGE Construction Métallique), 2 bureaux d'études privés (Arcadis ESG, Centre Technique Industriel de la Construction Métallique (CTICM)), 2 laboratoires publics (Institut Français des Sciences et Technologies des Transports, de l'Aménagement et des Réseaux (IFSTTAR) et l'Ecole Nationale des Travaux Publics de l'Etat (ENTPE)), 1 service technique du secteur public (Service d'Etudes des Transports, des Routes et leurs Aménagements (Sétra)-Coordinateur).

Budget : Le budget total du projet était de 2 369 000 € pour l'ensemble des partenaires dont 1 033 000 € pour EIFFAGE Travaux Publics. L'aide accordée à l'entreprise par l'ADEME pour le compte de l'ANR était de 297 000 € (cette aide comprenant un complément de financement de 12 000 € octroyé au titre du dispositif du Pôle de Compétitivité « Advancity – Ville et Mobilité Durable »).

Sujet : Ce projet propose de prendre en compte le comportement mécanique du revêtement des ponts à dalle métallique orthotrope, celui-ci devenant véritablement un élément de

structure, qu'il est donc naturel de chercher à optimiser. La conception d'une dalle orthotrope doit dès lors prendre en compte les paramètres suivants : la géométrie des augets, leur entraxe ainsi que celui des pièces de pont, l'épaisseur du platelage, mais également la nature du matériau constitutif du revêtement ainsi que son épaisseur. La conception intégrée du couple (dalle orthotrope, revêtement) constitue la grande nouveauté du projet ORTHOPLUS, dont le but est double : mise au point d'une part d'outils théoriques, méthodologiques et réglementaires permettant la prise en compte du couplage platelage-revêtement, pour le calcul de la durée de vie de la structure métallique, mais aussi du revêtement lui-même, et, d'autre part, développement d'une solution associant une dalle orthotrope et un revêtement en béton fibré ultra-performant (BFUP).

Résultats : Ma contribution porte sur les sous-projets suivants:

- Thème 1: Comportement mécanique de la dalle et du revêtement indépendants, en particulier le sous-thème 1d (Bilan des connaissances sur le comportement mécanique des revêtements bitumineux),
- Thème 2: Comportement de la dalle interagissant avec son revêtement, plus particulièrement les sous-thèmes 2b (Modèle tridimensionnel pour le calcul du revêtement), 2c (Mise au point d'un programme de validation expérimentale), 2d (Finalisation des modèles), 2e (Exploitation des modèles),
- Thème 3: Essais et mesures, en particulier les sous-thèmes 3b (Validation des modèles), 3c (Validation de la conception de la solution innovante), 3d (Mesures sur ouvrage réel: viaduc de Millau),
- Thème 4: Conception intégrée avec béton bitumineux, plus particulièrement les sous-thèmes 4a (Mise au point d'un modèle global simplifié), 4b (Optimisation de la structure), 4c (Analyse critique de l'essai de référence Sétra-LCPC),
- Thème 5: Conception intégrée avec béton fibré ultra-performant, en particulier le sous-thème 5c (Adaptation éventuelle de l'essai de référence Sétra-LCPC).

Ma contribution à ces différentes thématiques du projet ORTHOPLUS a été en grande partie réalisée via l'encadrement de la thèse en convention CIFRE (N° 2008/1199) de **S. Pouget**, intitulée « Influence des propriétés élastiques ou viscoélastiques des revêtements sur le comportement des ponts à dalle orthotrope », thèse ayant donné lieu à 6 publications scientifiques de rang A [17][18][22][23][24][25], 7 communications en congrès [45][62][63][72][73][74][75] et 2 articles de vulgarisation [106][114].

- **Projet RECYROUTE** (avril 2008-avril 2012), « Développement du béton de fraisats compacté fibré comme matériau composite durable pour assise de chaussée à fort ou très fort trafic ».

Thème : Le projet RECYROUTE a été lauréat de l'appel à projets de recherche du Réseau Génie Civil et Urbain (RGCU) lancé par l'Agence Nationale de Recherche (ANR) en 2007. L'objectif est de développer une nouvelle conception des dalles orthotropes et de leur revêtement, depuis les études théoriques aux applications techniques.

Partenaires : 3 entreprises privées (EIFFAGE Travaux Publics, Autoroutes Paris Rhin Rhone (APRR) et Chaussées Technique Innovation (CTI)), 2 laboratoires publics (Institut Français des Sciences et Technologies des Transports, de l'Aménagement et des Réseaux (IFSTTAR) et l'Ecole Nationale des Travaux Publics de l'Etat (ENTPE)), 1 service technique du secteur public (Pôle Scientifique et Technique (PST) de la Direction Régionale de l'Équipement d'Île-de-France (DREIF)) et 1 maîtrise d'ouvrage publique (Direction de la Voirie et des déplacements de la Ville de Paris).

Budget : Le budget total du projet était de 2 290 000 € pour l'ensemble des partenaires dont 510 300 € pour EIFFAGE Travaux Publics. L'aide accordée à l'entreprise par l'ADEME pour le compte de l'ANR était de 162 270 € (cette aide comprenant un complément de financement de 9 180 € octroyé au titre du dispositif du Pôle de Compétitivité « Advancity – Ville et Mobilité Durable »).

Sujet : Le recyclage des matériaux de chaussées est un enjeu majeur. La signature du 25 mars 2009 de la Convention d'engagement volontaire des acteurs de la profession routière française, en faveur notamment du recyclage dans les chaussées, concrétise des efforts entrepris en ce sens depuis de nombreuses années. Le recyclage des fraisats bitumineux

issus de la déconstruction routière est une pratique courante au sein d'enrobés bitumineux utilisant un bitume neuf d'apport. Le recyclage des fraisats dans un matériau composite traité au liant hydraulique est moins fréquent. Le comportement "composite" du matériau qui en découle est, de plus, moins bien cerné du fait du nombre réduit d'études aussi bien théoriques qu'expérimentales. Ajoutons que le guide de dimensionnement français des chaussées ne prend pas en compte le cas des matériaux composites.

Résultats : Ma contribution porte sur la direction administrative et scientifique de l'ensemble du projet. Néanmoins, ma contribution scientifique porte plus particulièrement sur les sous-projets suivants:

- Thème 1: Etat de l'art sur le comportement des matériaux traités aux liants hydrauliques d'une part et des enrobés bitumineux d'autre part, sur le comportement des matériaux fibrés, et enfin sur celui des premiers matériaux composites blanc-noir (liant hydraulique-liant bitumineux) de la littérature,
- Thème 2: Etude en laboratoire des propriétés des bétons de fraisats compactés fibrés, plus particulièrement les sous-thèmes 2-1 (Essais d'identification du gisement et des matériaux d'apport), 2-2 (Essais de formulation et essais mécaniques courants), 2-3 (Essais « avancés » : de module, de fatigue et de couplage thermo-mécanique (TSRST)),
- Thème 4: Etude de sections d'essais en béton de fraisats compacté fibré pour la modélisation, en particulier les sous-thèmes 4-1 (Observations des sections existantes), 4-3 (Section d'essai sur le manège de fatigue de l'IFSTTAR Nantes),
- Thème 5: Modélisation et dimensionnement,
- Thème 6: Chantier pilote sur aire de repos du réseau autoroutier APRR,
- Thème 8: Valorisation du projet.

Concernant les thèmes 1 et 2, ma contribution a été réalisée via l'encadrement de la thèse en convention CIFRE (N° 2009/0284) de **K. Bilodeau**, intitulée «Evaluation mécanique des matériaux composites incluant des fraisats bitumineux traités au liant hydraulique » et ayant donné lieu à 1 projet en cours de publication scientifique de rang A (Materials & Structures), 5 communications en congrès [54][64][67][68][76] et 1 article de vulgarisation [108].

#### Pilotage de projets collaboratifs avec fournisseurs

- **Collaboration avec BP (Beyond Petroleum) France et BP Europa**, sur l'étude en laboratoire des propriétés viscoélastiques linéaires et à la fatigue des liants et des enrobés bitumineux. Cette collaboration débutée courant 2010 fait l'objet du travail de thèse de S. Mangiafico, accueilli au laboratoire universitaire du Département Génie Civil et Bâtiment (DGCB) de l'ENTPE. Cette collaboration a donné lieu à 2 projets en cours de publications scientifiques de rang A [26][27] et 3 présentations en congrès [79][80][81].
- **Collaboration avec KRATON**, sur l'étude en laboratoire des propriétés à la rupture à très basses températures (-30°C et -35°C) des bitumes modifiés en élastomères: essai de propagation de fissure en mode I (ouverture) sur barreau préentaillé soumis à une flexion 3 points, calculs de la ténacité  $K_{Ic}$  et de l'énergie de rupture  $G_{Ic}$  (hypothèses de la Mécanique Linéaire de la Rupture), mise en évidence de niveaux de résistance très élevés pour les bitumes à phase continue en élastomère. Cette collaboration a donné lieu à 1 présentation en congrès [39].

D'autres projets collaboratifs, confidentiels, passés ou en cours, portent sur les additifs pour enrobés tièdes, ou encore sur les polymères alternatifs pour liants modifiés.

#### Pilotage de projets internes:

- Nombreuses **collaborations avec et entre les laboratoires centraux EIFFAGE Travaux Publics de Corbas (69) et de Ciry-Salsogne (02)**:
  - Formulations d'enrobés bitumineux acoustiques Nanophone<sup>®</sup>, de granulométrie 0/4mm [rapports internes confidentiels],

- Formulations et études d'optimisation d'enrobés à fort taux de recyclés RAFT<sup>®</sup> [41][52][102][112],
- Etudes sur les couplages thermo-mécaniques et la fissuration thermique à basse température des enrobés [4][34][36],
- Développement de l'essai de fatigue en flexion 5 points pour les revêtements de ponts à dalle orthotrope [10][11][35][88],
- Développement de l'essai de compression diamétrale pour la mesure du module complexe et du module sécant des enrobés [13][89][91],
- Développement de l'essai de module complexe et de fatigue sur liants et mastics bitumineux [90][100],
- Formulation d'enrobés selon des procédés d'enrobage semi-tièdes à 90°C, EBT<sup>®</sup> et EBE<sup>®</sup> [12][37][38][40][42][43][46][47][51][59][92][94][95][96][97][98][115][116],
- Formulation d'enrobés bitumineux à hautes performances et à granulométrie optimisée GB5<sup>®</sup> [19][21][54][55][56][58][65][66][104][107][109][111][113][117],
- Développement de liants bitumineux modifiés à phase continue polymère [70][71],
- Valorisation de sablons et de sables alluvionnaires à faible surface spécifique dans la formulation d'enrobés bitumineux [rapports internes confidentiels].

Pilotage et suivi de chantiers expérimentaux en collaboration avec le Sétra (Service d'Etudes sur les Transports, les Routes et leurs Aménagements)

- **Veille technologique Sétra:**

- Suivi à 5 et 10 ans d'un béton bitumineux semi-grenu (BBSG) de roulement à 20% de recyclés, sur autoroute A29 (réseau de la Société des Autoroutes du Nord et de l'Est de la France (SANEF)); Partenariat avec le Centre d'Études Techniques de l'Équipement (CETE) Nord-Picardie jugeant concluant ledit enrobé BBSG vis-à-vis du bon maintien dans le temps de ses propriétés mécaniques et d'adhérence par rapport aux caractéristiques d'un enrobé témoin sans agrégats d'enrobés recyclés [110].
- Suivi à 5 et 10 ans d'un enrobé à module élevé (EME) à 50% et 65% d'agrégats d'enrobés recyclés, sur autoroute A26 (réseau de la Société des Autoroutes du Nord et de l'Est de la France (SANEF)); Partenariat avec le Centre d'Études Techniques de l'Équipement (CETE) Nord-Picardie pour le suivi du maintien dans le temps des propriétés mécaniques de l'EME à fort taux de recyclés par rapport aux caractéristiques d'un enrobé témoin sans agrégats d'enrobés recyclés [en cours].

- **Charte de l'innovation autoroutière**, procédure conclue entre la Direction Générale des Infrastructures, des Transports et de la Mer (DGITM) et l'Association des Sociétés Françaises d'Autoroutes (ASFA):

- **Charte ASFA EBT<sup>®</sup>**: Préparation et soumission d'un dossier technique au Comité de Pilotage de l'ASFA ; Pilotage de deux expérimentations sur les réseaux de la société SANEF et de la société des Autoroutes en Région Rhône Alpes (AREA) ; Suivi sur 5 ans en partenariat avec les services et établissements du réseau scientifique et technique du ministère chargé du développement durable, en particulier les CETE Nord-Picardie et de Lyon jugeant concluant la technique EBT<sup>®</sup> vis-à-vis de l'objectif de "réduction des émissions dans l'atmosphère et de la consommation d'énergie", avec un bon maintien dans le temps des propriétés mécaniques par rapport aux caractéristiques d'un enrobé équivalent à chaud ; Certificat de bonne fin délivré fin 2012 par E. LE GUERN, Directeur du Sétra.
- **Charte ASFA BBTM Orthoprène<sup>®</sup>**: Préparation et soumission d'un dossier technique au Comité de Pilotage de l'ASFA ; Recherche en cours de sites d'expérimentation sur les réseaux autoroutiers concédés, avec pour objectif la validation de la pérennité des caractéristiques mécaniques, d'adhérence et de résistance au roulement d'un Béton Bitumineux Très Mince (BBTM) au liant modifié Orthoprène<sup>®</sup> (liant bitumineux très modifié, à phase continue polymère) ; Suivi prévu par les CETE et Sétra.

- **Dispositif de soutien à l'innovation « routes et rues »**. Cette procédure est destinée à expérimenter des innovations dans le domaine de la voirie et des réseaux divers, et à en assurer l'évaluation, en s'appuyant sur un programme public national consacré à l'innovation

routière qui repose sur une procédure de sélection annuelle. L'ensemble de la procédure s'inscrit dans le cadre de l'article 75 du code des marchés publics.

- **Projet EFEME** (Enrobés à Forte Economie en Matériaux et en Energie) lauréat de l'appel à projets innovants de l'année 2007 ; Réalisation et suivi de deux chantiers expérimentaux d'enrobés semi-tèdes EBT<sup>®</sup> incorporant des forts taux de recyclés (RN10 pour le compte de la Direction interdépartementale des routes du Nord-Ouest et RD817 pour le compte du Conseil Général de Haute Garonne) en partenariat avec les CETE Normandie Centre et du Sud-Ouest [105].
- **Projet STGV** (Semi-Tiède à Grande Vitesse) lauréat de l'appel à projets innovants de l'année 2007 ; Réalisation d'un chantier expérimental d'enrobé semi-tède EBT<sup>®</sup> avec un finisseur à grande cadence (RN94, réseau de la Direction interdépartementale des routes Méditerranée) et suivi en partenariat avec le CETE Méditerranée.
- **Projet GB5<sup>®</sup>** lauréat de l'appel à projets innovants de l'année 2010 ; Réalisation de cinq chantiers expérimentaux de grave-bitume à hautes performances GB5<sup>®</sup>, trois dans le cadre de la procédure nationale de soutien à l'innovation « routes et rues » (A813-SANEF, A41N-AREA, RD1324-Conseil Général de l'Oise) et deux dans le cadre de veille technologique Sétra (RD101- Conseil Général de la Vendée, RD612- Conseil Général de l'Hérault) ; Suivi en partenariat avec respectivement les CETE Normandie-Cente, de Lyon, Nord-Picardie, Ouest et Méditerranée ; Objectif de validation de la pérennité d'enrobés d'assise à granulométrie optimisée (à simple ou double discontinuité) et aux liants de types multigrade ou modifiés en polymères.
- **Projet BB5<sup>®</sup>** lauréat de l'appel à projets innovants de l'année 2012 ; 3 chantiers expérimentaux seront réalisés en 2013 et suivis pendant 3 années au minimum ; Objectif de validation de la pérennité d'enrobés de roulement à granulométrie optimisée, dense et grenue, et sans reprofilage préalable ; Technique à la fois gage de durabilité mais aussi sobre et économique.

### 2.2.3 Partenariats scientifiques

#### Projets de coopération internationale RILEM (Réunion Internationale des Laboratoires et Experts des Matériaux, systèmes de construction et ouvrages)

- **Technical Committee 206-ATB** (2007-2011), Advanced testing and characterization of bituminous materials, Technical Group 3: Mechanical testing of mixtures, French wheel tracking round robin test on a polymer modified bituminous mixture.

Coordinateur: H. Di Benedetto (ENTPE).

Partenaires: France (ENTPE, IFSTTAR, EIFFAGE Travaux Publics, TOTAL), Royaume-Uni (Université de Nottingham-NTEC), Pologne (IBDiM), Belgique (BRRC), Québec (ETS).

Sujet: Essai croisé organisé pour évaluer la reproductibilité de l'essai d'orniérage français à 50°C et 60°C, sur un enrobé utilisant un liant modifié en polymère.

Résultats: Deux problèmes avaient auparavant été reportés dans la littérature: échauffement excessif de l'échantillon dû à la friction et collage du bitume à la roue de l'orniéreur. Deux procédures ont été proposées puis évaluées par le groupe pour améliorer la reproductibilité de l'essai. Les différents pneus utilisés pour la roue de l'orniéreur, ainsi que la surcompacité de certains échantillons, sont les deux principales causes identifiées de dispersion des essais. Cette collaboration a donné lieu à 1 publication scientifique de rang A [20].

- **Technical Committee 237-SIB** (2011-2014), Testing and characterization of sustainable innovative bituminous materials and systems, Technical Group 3: 3-dimensional characterization of mixtures.

Coordinateur: H. Di Benedetto (ENTPE).

Partenaires: France (ENTPE, Université d'Egletons, IFSTTAR, EIFFAGE Travaux Publics, TOTAL), Autriche (Université de Vienne), Italie (Université de Parme), Royaume-Uni (Université de Nottingham), Pologne (IBDiM), Belgique (BRRC), Suisse (EMPA), Québec (ETS).

Sujet: Développement d'une caractérisation expérimentale et d'une modélisation 3D de 2 enrobés bitumineux, GB3 et GB5<sup>®</sup>, fabriqués par EIFFAGE Travaux Publics puis répartis dans l'ensemble des laboratoires partenaires. Viscoélasticité linéaire : mesures de  $E^*$  et  $\nu^*$  selon plusieurs directions, modélisation.

Résultats: en cours.

- **Collaboration scientifique avec l'IFSTTAR**, formalisée par le biais d'un contrat-cadre signé en 2005 entre les Directions Générales de l'IFSTTAR et d'EIFFAGE Travaux Publics. Depuis, 5 avenants à caractère technique et environnemental ont été signés, 4 avec l'ex-Division Structures et Matériaux pour les Infrastructures de Transport (SMIT), désormais à l'origine pour partie de l'actuel Département Matériaux (MAT) de l'IFSTTAR, et 1 avec l'actuel Groupe MIT (Matériaux pour les Infrastructures de Transport) du Département MAT:

- **2005**: 1<sup>er</sup> avenant à la convention-cadre, intitulé « **Enrobés tièdes à base de mousse** ». Ce projet, co-encadré par C. de la Roche et B. Cazacliu (50%) et moi-même (50%), porte sur la faisabilité de procédés d'enrobage de granulats à 20°C, 70°C et 90°C avec une mousse de bitume.

Cette collaboration a donné lieu à 1 publication scientifique de rang A [15] et 1 publication au sein d'une revue française à comité de lecture [84].

- **2006**: 2<sup>e</sup> avenant, intitulé « **Enrobés à Basse Température EBT<sup>®</sup>** ». Ce projet, co-encadré par C. de la Roche et V. Gaudefroy (50%) et moi-même (50%), a pour objectif d'évaluer les performances des enrobés EBT<sup>®</sup>, d'une part, et d'apprécier la validité de la méthodologie de la formulation des enrobés à chaud, d'autre part.
- **2007**: 3<sup>e</sup> avenant à la convention-cadre, intitulé « **Formulation des EBT<sup>®</sup>** », qui a pour but de compléter les travaux de l'avenant n°2 (2006). Ce projet, co-encadré par C. de la Roche et V. Gaudefroy (50%), d'une part, et par E. Beduneau et moi-même (50%), d'autre part, a donné lieu à 1 communication en conférence internationale [60] et 1 publication au sein d'une revue professionnelle [101].
- **2007**: 4<sup>e</sup> avenant, intitulé « **Evaluation environnementale des EBT<sup>®</sup>** ». Ce projet, co-encadré par C. de la Roche et V. Gaudefroy (50%) et par E. Beduneau et moi-même (50%), porte sur l'évaluation en laboratoire des émissions générées par la fabrication des enrobés semi-tièdes EBT<sup>®</sup> en fonction du liant utilisé, de la teneur en eau et des additifs utilisés, en comparaison d'une formule témoin d'enrobé à chaud.

Cette collaboration a donné lieu à 4 communications en conférences internationales [44][48][49][60].

- **2012** (en cours): 5<sup>e</sup> avenant à la convention-cadre, intitulé « **Relation entre les performances mécaniques et la structure interne de la grave-bitume GB5<sup>®</sup>** ». Ce projet, co-encadré par F. Hammoum et O. Burban (50%), et par S. Pouget et moi-même (50%), a pour objet l'évaluation de :
  - i) la pertinence de l'analyse d'images afin d'étudier les relations entre structure interne des enrobés (au travers de quantificateurs géométriques) et performances mécaniques ;
  - ii) la maniabilité des enrobés (via la mesure du pseudo-cisaillement à la presse à cisaillement giratoire (PCG)) ;
  - iii) l'émission de composés organiques volatils (COV) à la fabrication des enrobés à granulométrie optimisée GB5<sup>®</sup>.

Par ailleurs, entre 2007 et 2008, 3 séries de mesures d'émissions et de consommations en centrales d'enrobage ont été réalisées en collaboration avec l'ex-Division Démarches durables en Génie Civil (DDGC, dirigée alors par P. Tamagny et A. Jullien), désormais à l'origine pour partie de l'actuel Département MAT:

- **2007** : **Inventaire du cycle de vie** réalisé dans le cadre du chantier de la RD231 (Conseil Général de Seine et Marne) et de la fabrication d'enrobés semi-tièdes EBT<sup>®</sup> sur le poste de Monthyon (EIFFAGE Travaux Publics Centre Ile-de-France).

A donné lieu à 2 communications en conférences internationales [50][61] et 1 publication au sein d'une revue professionnelle [103].



- **2008 : Inventaire du cycle de vie** réalisé dans le cadre du chantier de la RD817 (Conseil Général de Haute Garonne) et de la fabrication d'enrobés semi-tièdes EBT<sup>®</sup> sur le poste de SMEG (EIFFAGE Travaux Publics Sud-Ouest).

A donné lieu à 1 communication en conférence internationale [61].

- **2008 : Inventaire du cycle de vie** réalisé dans le cadre d'une planche expérimentale sur autoroute A5 (réseau concédé APRR) de grave-bitume semi-tiède EBT<sup>®</sup> fabriquée sur une centrale mobile EIFFAGE Travaux Publics Grands Travaux.

- **ETS:**

- Collaboration avec le **Prof. A Carter** sur la méthodologie de fabrication en laboratoire d'enrobés semi-tièdes EBT<sup>®</sup> à 90°C ; réalisation d'une évaluation performancielle d'EBT<sup>®</sup> avec matériaux recyclés au laboratoire de l'ETS. Collaboration ayant donné lieu à 1 publication en congrès international [69].
- Collaboration avec le **Prof. D. Perraton** portant sur les méthodes d'optimisation des empilements granulaires pour réaliser des enrobés à la fois maniables, à grande aptitude au compactage, tout en conservant une bonne résistance à l'orniérage. Collaboration ayant donné lieu à 1 publication scientifique de rang A [16] et 1 publication en congrès international [53].

#### 2.2.4 Brevets

- Invention du brevet **FR0503527** (priorité 08.04.2005) [115], avec ses extensions internationales **EP1717369**, **US20060236614**, **CA2541622**, **ZA200602861**, **NZ546419**, **AU2006201444**, **MA28260**, **AR56965**, **BR200601348**.

L'invention propose un procédé pour fabriquer un enrobé tiède (100°C à 140°C) ou semi-tiède (<100°C) comprenant des fragments solides enrobés d'un liant, le procédé comprenant une étape de séchage des fragments solides suivie d'une étape d'enrobage par adjonction exclusivement d'un liant à la totalité des fragments solides, caractérisé en ce que l'étape de séchage est réalisée dans des conditions laissant subsister une fraction de l'humidité initiale des fragments solides.

- Invention du brevet divisionnaire **FR0507655** (priorité 08.04.2005) [116], avec ses extensions internationales **EP1712680**, **US20060240185**, **CA2541625**, **ZA200602862**, **NZ546420**, **AU2006201443**, **MA28261**, **AR55907**, **BR200601349**.

L'invention propose un procédé pour fabriquer un enrobé tiède (100°C à 140°C) ou semi-tiède (<100°C) comprenant des fragments solides enrobés d'un liant, dans lequel on sèche et on enrobe une première partie des fragments solides après quoi la partie restante des fragments solides, ayant conservé son humidité initiale, est ajoutée au mélange obtenu, caractérisé en ce qu'au moins ladite première partie des fragments solides contient des fines.

- Invention du brevet **FR1052595** (priorité 06.04.2010) [117], avec son extension internationale PCT **WO2011124799**.

L'invention propose un enrobé bitumineux à hautes performances pour couche d'assise ou de liaison de chaussée routière ou autoroutière, ou pour plates-formes industrielle, portuaire, ou aéroportuaire, ou encore pour couche de support de voie ferroviaire. La structure granulaire est optimisée, très dense et telle qu'elle comprend une voire plusieurs discontinuités granulaires, tandis que la teneur en bitume représente au plus 5% en poids de l'enrobé. Par ailleurs ; le bitume est modifié par inclusion de polymères et/ou d'huile, et/ou traité par soufflage et/ou traité par moussage ou par émulsion, moyennant quoi les propriétés de module de rigidité et de tenue en fatigue de l'enrobé sont exceptionnelles.

- Collaboration avec la Direction Matériel EIFFAGE Travaux Publics pour la préparation du dépôt de brevet **FR2902117** avec son extension européenne **EP1867784**.

L'invention propose un dispositif pour fabriquer un enrobé bitumineux, comprenant au moins un poste de malaxage des granulats, qui comprend en outre un poste de traitement de dégagements gazeux (par effet Venturi) relié au poste de malaxage et agencé de manière à capter, au moins en partie, lesdits dégagements gazeux et à séparer une partie des

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substances en suspension dans lesdits dégagements gazeux. En particulier, le poste de traitement de dégagements gazeux comprend un organe de lavage de gaz agencé de manière à mettre en contact un liquide de lavage avec des dégagements gazeux, ainsi qu'un organe de décantation agencé de manière à récupérer au moins en partie le produit de la mise en contact du liquide de lavage et des dégagements gazeux.

### 2.2.5 Expertise d'articles

2004-présent: Reviewer pour les revues **Road Materials and Pavement Design** (Taylor & Francis), **Materials & Structures** (Rilem), **Materials in Civil Engineering & Transportation Engineering** (Am. Society of Civil Engineers).

### 2.2.6 Jurys de thèses

Membre de plusieurs jurys de soutenance de thèses (hormis les doctorants co-encadrés), notamment:

- **M.L. Nguyen**, Etude de la fissuration et de la fatigue des enrobés bitumineux, Institut National des Sciences Appliquées de Lyon - Ecole Nationale des Travaux Publics de l'Etat, Université de Lyon, 2009.
- **P. Clec'h**, Comportement des enrobés bitumineux sous sollicitation multidirectionnelle, Institut National des Sciences Appliquées de Lyon - Ecole Nationale des Travaux Publics de l'Etat, Université de Lyon, 2010.
- **S.C. Somé**, Comportement thermomécanique des enrobés tièdes et de l'interface bitume-granulat, École Polytechnique de l'Université de Nantes, 2012.

### 2.2.7 Affiliations à des comités d'experts

2004-présent: Membre de l'Editorial Board de l'International Journal of Road Materials & Pavement Design (Taylor & Francis).

2008-présent: Membre du Comité Scientifique des conférences bisannuelles de l'EATA (European Asphalt Technologists Association).

2012-présent: Membre du Conseil Stratégique de l'Institut des Procédés Appliqués aux Matériaux (IPAM) de l'Université de Limoges.



# Chapitre III. Liste des publications et travaux

## DEA (Diplôme d'Etudes Approfondies) en Génie Civil

- [1] **F. Olard**, Etude et modélisation de comportement thermo-mécanique des enrobés bitumineux, Mémoire de recherche pour l'obtention du DEA de Génie Civil, ENTPE, juillet 2000.

## Thèse de Doctorat en Génie Civil

- [2] **F. Olard**, Comportement thermomécanique des enrobés bitumineux à basses températures. Relations entre les propriétés du liant et de l'enrobé. Thèse de Doctorat en Génie Civil ENTPE-INSA de Lyon, octobre 2003.

## Chapitres d'ouvrages scientifiques (2)

- [3] H. Di Benedetto & **F. Olard**, DBN Law for the Thermo-Visco-Elasto-Plastic Behavior of Asphalt Concrete, Chapter 9 in Modeling of Asphalt Concrete, pp 245-268, Editor-in-chief Y. Richard Kim, New York: McGraw-Hill Professional, ASCE Press, 2009.
- [4] **F. Olard**, Prediction of the thermal cracking resistance of bituminous binders and mixtures, Essay in Chapter D of Asphalt material behaviour at low temperatures, Essays in honour of Prof. W. Arand on the occasion of his 80<sup>th</sup> birthday, pp 237-253, Editor-in-chief Michael P. Wistuba, Institut für Straßenwesen at Braunschweig University, ISBN 3-932164-11-3, 2009.

## Publications au sein de revues internationales à comité de lecture<sup>3</sup> (23)

- [5] **F. Olard**, H. Di Benedetto, B. Eckmann, JP. Triquigneaux, Linear viscoelastic properties of bituminous binders and mixtures at low and intermediate temperatures, Road Materials and Pavement Design, Volume 4, Issue 1, 2003.
- [6] **F. Olard**, H. Di Benedetto, General "2S2P1D" model and relation between the linear viscoelastic behaviors of bituminous binders and mixes, Road Materials and Pavement Design, Volume 4, Issue 2, 2003.
- [7] H. Di Benedetto, **F. Olard**, C. Sauzéat, B. Delaporte, Linear viscoelastic behavior of bituminous materials: from binders to mixes, 1<sup>st</sup> EATA (European Asphalt Technology Association) Congress, Nottingham, Special Issue of Road Materials and Pavement Design, 2004.
- [8] **F. Olard**, H. Di Benedetto, The "DBN" Model: A Thermo-Visco-Elasto-Plastic Approach for Pavement Behavior Modeling. Application to Direct Tension Test and Thermal Stress Restrained Specimen Test, Journal of the Association of Asphalt Paving Technologists (AAPT), Vol.74, pp.791-828, 2005.
- [9] **F. Olard**, H. Di Benedetto, A. Dony, JC. Vaniscote, Properties of bituminous mixtures at low temperatures and relations with binder characteristics, Materials and structures, Vol.38, pp.121-126, 2005.

<sup>3</sup> Publications [5-8] non référencées WoS : *i)* revue RMPD référencée WoS à partir de 2005 et *ii)* Journal AAPT donnant lieu à un Special Issue RMPD à partir de 2012.

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- [10] **F. Olard**, B. Hérítier, F. Loup, S. Krafft, New French Standard Test Method for the Design of Surfacing on Steel Deck Bridges. Case Study of the Millau Viaduct, *Road Materials and Pavement Design*, Vol. 6, Issue 4 - pp.515-531, 2005.
- [11] B. Hérítier, **F. Olard**, F. Loup, S. Krafft, Design of a Specific Bituminous Surfacing for the World's Highest Orthotropic Steel Deck Bridge, *Transportation Research Record: Journal of the Transportation Research Board*, No. 1929, Transportation Research Board of the National Academies, Washington D.C., pp. 141-148, 2005.
- [12] Romier, M. Audeon, J. David, Y. Martineau, **F. Olard**, Low-Energy Asphalt with Performance of Hot-Mix Asphalt, *Transportation Research Record: Journal of the Transportation Research Board*, No. 1962, Transportation Research Board of the National Academies, Washington D.C., pp. 101-112, 2006.
- [13] **F. Olard**, F. Noël, F. Loup, Evaluation of modulus testing in indirect tension mode, *Road Materials and Pavement Design*, Volume 7, Issue 4 - pp.543-554, 2006.
- [14] H. Di Benedetto, M. Neifar, C. Sauzéat, **F. Olard**, Three-dimensional thermo-viscoplastic behaviour of bituminous materials: the DBN model, *Road Materials and Pavement Design*, Vol.8, Issue 2, pp.285-316, 2007.
- [15] V. Gaudefroy, **F. Olard**, B. Cazacliu, C. de La Roche, E. Beduneau, JP. Antoine, Laboratory investigations of mechanical performance of foamed bitumen mixes that use half-warm aggregates, *Transportation Research Record: Journal of the Transportation Research Board*, No. 1998, Transportation Research Board of the National Academies, Washington DC, pp. 89-95, 2007.
- [16] **F. Olard**, D. Perraton, On the optimization of the aggregate packing characteristics for the design of high-performance asphalt concretes, 4<sup>th</sup> EATA (European Asphalt Technology Association) Congress, Parma, Special Issue of *Road Materials and Pavement Design*, 2010.
- [17] S. Pouget, C. Sauzéat, H. Di Benedetto, **F. Olard**, Numerical simulation of the five-point bending test designed to study bituminous wearing courses on orthotropic steel bridge, *Materials and structures*, Vol. 43, n°3, pp. 319-330, 2010.
- [18] S. Pouget, C. Sauzéat, H. Di Benedetto, **F. Olard**, From the behavior of constituent materials to the calculation and design of orthotropic bridge structures, 4<sup>th</sup> EATA (European Asphalt Technology Association) Congress, Parma, Special Issue of *Road Materials and Pavement Design*, 2010.
- [19] **F. Olard**, GB5 mix design: high-performance & cost-effective asphalt concretes by use of gap-graded curves & SBS modified bitumens, Annual meeting of AAPT (Association of Asphalt Paving Technologists), Austin (Texas), Special Issue of *Road Materials and Pavement Design*, Vol. 13, Supplement 1, pp. 234-259, 2012.
- [20] T. Gabet, H. Di Benedetto, D. Perraton, J. De Visscher, T. Gallet, W. Bankowski, **F. Olard**, J. Grenfell, D. Bodin, C. Sauzéat, French wheel tracking round robin test on a polymer modified bitumen mixture (RILEM TC 206-ATB, TG3: Mechanical testing of mixtures), *Materials and structures*, ISSN 1359-5997 , Vol. 44, N° 6, pp. 1031-1046, 2011.
- [21] **F. Olard**, GB5<sup>®</sup>: an alternative to EME2 for long-lasting and cost-effective asphalt concretes by use of aggregate packing & SBS modified binders, *The International Journal of Pavement Engineering & Asphalt Technology*, ISSN 1464-8164, Vol. 13, Issue 1, pp. 22-37, 2012.
- [22] S. Pouget, C. Sauzéat, H. Di Benedetto, **F. Olard**, Effect of Vehicle Speed on the Millau Viaduct Response, *Journal of Testing and Evaluation (JOTE)*, Vol. 40, N° 7, 2012.
- [23] S. Pouget, C. Sauzéat, H. Di Benedetto, **F. Olard**, Viscous energy dissipation in asphalt pavement structures and implication on vehicle fuel consumption, *Journal of Materials in Civil Engineering (JMCE)*, Vol. 24, Issue 5, pp. 568-576, 2012.
- [24] S. Pouget., C. Sauzéat, H. Di Benedetto, **F. Olard**, Modeling of viscous bituminous wearing course materials on orthotropic steel deck, *Materials and structures*, ISSN 1359-5997, Vol. 45, N° 7, pp. 1115-1125, 2012.
- [25] S. Pouget., C. Sauzéat, H. Di Benedetto, **F. Olard**, Calculation of viscous energy dissipation in asphalt pavements, *Baltic Journal of Road and Bridge Engineering*, 2012. [accepted]

- [26] S. Mangiafico, H. Di Benedetto, C. Sauzéat, **F. Olard**, S. Pouget, S. Dupriet, L. Planque, R. Van Rooijen, Influence of RAP and Asphalt Mix Composition on Mechanical Properties of Asphalt Mixes, Annual meeting of AAPT (Association of Asphalt Paving Technologists), Denver (Colorado), Special Issue of Road Materials and Pavement Design, Vol. 14, 2013 [Soumission].
- [27] S. Mangiafico, H. Di Benedetto, C. Sauzéat, **F. Olard**, S. Pouget, L. Planque, Influence of RAP content on complex modulus of asphalt binder blends and corresponding mixes: experimental results and modeling, Annual meeting of EATA (European Asphalt Technologists Association), Braunschweig (Germany), Special Issue of Road Materials and Pavement Design, Vol. 14, 2013 [Soumission].

**Publications en conférences internationales et nationales à comité de lecture (présentations & proceedings) (54)**

- [28] H. Di Benedetto, M. Neifar, B. Dongmo, **F. Olard**, "Loi thermo-viscoplastique pour les mélanges bitumineux : simulation de la perte de linéarité et du retrait empêché", 36<sup>ème</sup> Colloque Annuel du Groupe Français de Rhéologie, Marne-la-Vallée, 2001.
- [29] **F. Olard**, H. Di Benedetto, A. Dony, JC. Vaniscote, Properties of bituminous mixtures at low temperatures and relations with binder characteristics, 6<sup>th</sup> International RILEM Symposium on PTEBM, Zurich, 2003.
- [30] **F. Olard**, H. Di Benedetto, B. Eckmann, JC. Vaniscote, Low-temperature failure behavior of bituminous binders and mixes, Transportation Research Board meeting, Washington D.C., 2004.
- [31] **F. Olard**, H. Di Benedetto, Fracture toughness and fracture energy of bituminous binders at low temperatures, 5<sup>th</sup> RILEM International Conference "Cracking in Pavements", Limoges, 2004.
- [32] **F. Olard**, H. Di Benedetto, JP. Triquigneaux, M. Mazé, Thermal cracking of bituminous mixtures: Experimentation and modeling, 3<sup>rd</sup> Eurobitume & Eurasphalt Congress, Vienne, 2004.
- [33] **F. Olard**, H. Di Benedetto, JC. Vaniscote, B. Eckmann, Failure behavior of bituminous binders and mixes at low temperatures, 3<sup>rd</sup> Eurobitume & Eurasphalt Congress, Vienne, 2004.
- [34] **F. Olard**, JC. Vaniscote, F. Loup, Prediction of thermal cracking resistance of some SBS modified binders and mixes, 7<sup>th</sup> International Conference on Bearing Capacity of Roads and Airfields (BCRA) congress, Trondheim, 2005.
- [35] B. Héritier, **F. Olard**, M. Saubot, S. Krafft, Design of a specific bituminous surfacing for orthotropic steel bridge decks: application to the Millau Viaduct, 7<sup>th</sup> International Conference on Bearing Capacity of Roads and Airfields (BCRA) congress, Trondheim, 2005.
- [36] **F. Olard**, F. Loup, Characterization of Thermal Cracking Resistance of Bituminous Mixtures Using Monotonic and Cyclic Restrained Cooling Tests, 10<sup>th</sup> International Conference on Asphalt Pavements, Quebec City, 2006.
- [37] **F. Olard**, C. Le Noan, A. Romier, Technique innovante d'enrobage à basse énergie pour minimiser les impacts environnementaux de la centrale d'enrobage au chantier / Innovative low energy asphalt technique for minimizing impacts from asphalt plants to road works, World Road Association congress (PIARC), Paris, 2007.
- [38] **F. Olard**, JP. Antoine, B. Héritier, A. Romier, Y. Martineau, LEA (low energy asphalt): a new generation of half-warm asphalt mixtures, International Conference on Advanced Characterization of Pavement and Soil Engineering Materials, Athens, 2007.
- [39] **F. Olard**, E.J. Scholten, R. Kluttz, Gaining Experience in Applying Fracture Toughness Tests for Asphalt Binders at Low Temperatures, 6<sup>th</sup> International Symposium on Binder Rheology and Pavement Performance, Tampa (Florida), 2007.
- [40] **F. Olard**, C. Le Noan, E. Beduneau, A. Romier, Low Energy Asphalts for Sustainable Road Construction, 4<sup>th</sup> Eurasphalt & Eurobitume Congress, Copenhagen, 2008.

- [41] **F. Olard**, C. Le Noan, D. Bonneau, S. Dupriet, C. Alvarez, Very high recycling rate (>50%) in hot mix and warm mix asphalts for sustainable road construction, 4<sup>th</sup> Eurasphalt & Eurobitume Congress, Copenhagen, 2008.
- [42] **F. Olard**, A. Romier, B. Hérítier, Y. Martineau, E. Beduneau, Low energy asphalt LEA<sup>®</sup>: new half-warm mix asphalt for minimizing impacts from asphalt plant to job site, ISAP Congress on Asphalt Pavements and Environment, Zurich, 2008.
- [43] C. Sauzéat, H. Di Benedetto, **F. Olard**, ML. Nguyen, Fatigue behaviour of half-warm mix asphalts, ISAP Congress on Asphalt Pavements and Environment, Zurich, 2008.
- [44] V. Gaudefroy, **F. Olard**, C. de la Roche, JP. Antoine, E. Beduneau, Laboratory investigations on the Total Organic Compounds emissions of half-warm mix asphalt technology versus traditional hot mix asphalt, ISAP Congress on Asphalt Pavements and Environment, Zurich, 2008.
- [45] S. Pouget, C. Sauzéat, H. Di Benedetto, **F. Olard**, Wearing course influence on the orthotropic steel bridge deck behavior from numerical simulations of the five-point bending test, International Orthotropic Bridge Conference (IOBC), Sacramento (California), 2008.
- [46] **F. Olard**, E. Beduneau, D. Bonneau, S. Dupriet, N. Seignez, Laboratory characterization of half-warm mix asphalts with high recycling rate by means of the factorial experiment design approach, 8<sup>th</sup> International Conference on Bearing Capacity of Roads and Airfields (BCRA) congress, Champaign (Illinois), 2009.
- [47] **F. Olard**, E. Beduneau, N. Seignez, S. Dupriet, D. Bonneau, Laboratory Performance-Based Assessment of Half-Warm Mix Asphalts with High Recycling Rate by means of the Factorial Experiment Design Approach, 7<sup>th</sup> Int. RILEM Symposium on Advanced Testing and Characterization of Bituminous Materials (ATCBM), Rhodos, 2009.
- [48] V. Gaudefroy, **F. Olard**, E. Beduneau, C. de la Roche, Influence of the low-emission asphalt LEA<sup>®</sup> composition on total organic compounds emissions using the factorial experimental design approach, ENVIROAD Congress, Warsaw, 2009.
- [49] V. Gaudefroy, **F. Olard**, M. Ridane, E. Beduneau, C. de la Roche, Laboratory environmental assessment of half-warm mix asphalts by means of the factorial experiment design approach, Transportation Research Board annual meeting, Washington D.C., 2009.
- [50] A. Ventura, P. Monéron, A. Jullien, P. Tamagny, **F. Olard**, D. Zavan, Environmental comparison at industrial scale of hot and half-warm mix asphalt manufacturing processes, Transportation Research Board annual meeting, Washington D.C., 2009.
- [51] **F. Olard**, E. Beduneau, D. Fallone, L. Miranda Perez, Low energy & low emission asphalts for sustainable road construction, 16<sup>th</sup> International Road Federation World Meeting, Lisbon, 2010.
- [52] **F. Olard**, C. Le Noan, D. Bonneau, M. Jourdan, R. Cherfils, Very high recycling rate (up to 70%) in hot mix and warm mix asphalts for sustainable road construction, 16<sup>th</sup> International Road Federation World Meeting, Lisbon, 2010.
- [53] **F. Olard**, P. Huon, D. Perraton, C. Billet, On the optimization of the aggregate packing for the design of self-blocking high-performance asphalts, 11<sup>th</sup> International Conference on Asphalt Pavements, Nagoya, 2010.
- [54] K. Bilodeau, C. Sauzéat, H. Di Benedetto, **F. Olard**, D. Bonneau, Mechanical behavior of fiber reinforced compacted concrete with incorporation of reclaimed asphalt pavement, 11<sup>th</sup> International Conference on Asphalt Pavements, Nagoya, 2010.
- [55] **F. Olard**, Innovative Design Approach for High-Performance Asphalt Concretes for Long-Life Base and Binder Courses by Use of Aggregate Packing Concepts and Polymer Modified Binders, Transportation Research Board annual meeting, Washington D.C., 2011.
- [56] **F. Olard**, Innovative Design of the So-Called GB5<sup>®</sup> High-Performance Asphalt Mixtures for Long-Life Pavements, World Road Association congress (PIARC), Mexico City, 2011.
- [57] **F. Olard**, GB5: innovative design of high-performance asphalt mixes for long-life & cost effective pavements by optimizing aggregates & using SBS modified bitumens, 10<sup>th</sup> Conference on Asphalt Pavements for Southern Africa, KwaZulu-Natal, South Africa, 2011.

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# Chapitre IV. Travaux de recherche et de diffusion scientifique

## 4.0 Introduction

### 4.0.1 Contexte de l'industrie routière et besoins de recherche associés

Dès les années 1970, la France adoptait une démarche de dimensionnement des chaussées fondée sur les outils de la mécanique rationnelle. Un guide technique de conception des structures de chaussée a notamment été élaboré (Sétra-LCPC, 1994 et 1997). Ce guide constitue un cadre de référence technique unitaire permettant d'aborder la définition des différentes structures de chaussée en fonction de la stratégie d'investissement et de gestion retenue par chaque maître d'ouvrage.

Récemment, un système performantiel comprenant méthodes d'essai, équipement et spécifications a été élaboré suite au programme de recherche américain SHRP (Strategic Highway Research Program) entre 1987 et 1993. 50 millions de dollars (soit 1/3 du budget total du SHRP) ont été attribués à la recherche consacrée aux matériaux bitumineux. Ce système performantiel, dénommé SUPERPAVE (Superior Performing Asphalt Pavements), fait actuellement l'objet de compléments.

Les techniques routières française et américaine sont toutes deux complémentaires, rationnelles et de haut niveau, si bien que leur diffusion à l'échelle internationale s'en trouve facilitée. En conséquence, bon nombre de pays adoptent les règles française et américaine de formulation et de calculs des structures de chaussées. Ces deux approches techniques sont toutefois confrontées à une recrudescence de nouveaux matériaux (bitumes rectifiés à l'air, bitumes modifiés en polymères, etc.) et à un recours plus systématique au recyclage, nécessitant quelques améliorations et une approche plus fondamentale. En outre, les besoins de recherche associés à l'industrie routière se doivent de présenter à la fois un volet technique –bien sûr–, mais aussi un volet environnemental et un volet économique.

Le volet environnemental peut se décliner de la façon suivante:

- Diminution du prélèvement de ressources non renouvelables nobles ou sensibles ;
- Usage de matériaux à faible bilan carbone sur leur cycle de vie ;
- Valorisation de déchets du BTP et de sous-produits industriels ;
- Recours aux matériaux renouvelables.

Dans un contexte financier contraint, mais où néanmoins les infrastructures routières vieillissent, le volet économique peut se décliner quant à lui de la façon suivante:

- Recours aux techniques sobres, d'entretien facile et de faible coût ;
- Tout en garantissant une bonne durabilité et toutes les garanties en terme de sécurité ;
- Bon rapport (réponse au besoin) / (qualité) / (prix).

### 4.0.2 Démarche personnelle et contributions scientifiques

Une particularité de mon parcours et de mes travaux de recherche tient à mon statut de responsable de projets R&D en entreprise. Celui-ci suppose, en effet, d'avoir des compétences scientifiques et techniques, en recherche appliquée mais aussi fondamentale, afin de participer à l'évolution indispensable du savoir-faire en formulation et en production des matériaux. Cela implique d'avoir des connaissances dans les deux domaines et de mettre en adéquation le savoir et le besoin industriel. De cet échange entre recherches appliquée et fondamentale, sont nées nombre d'innovations.

Sans être pour autant exhaustif, ce chapitre présente mes principaux travaux de recherche et de diffusion scientifique, depuis ma thèse jusqu'à aujourd'hui, dont le fil conducteur apparaît en filigrane: **l'optimisation des performances des matériaux bitumineux par une meilleure compréhension des relations entre leur composition, leur structure et leurs propriétés thermo-mécaniques.**

Les sections suivantes présentent plus spécifiquement mes contributions à six thématiques majeures :

- Section 4.1 : Comportement thermo-mécanique des enrobés bitumineux à basses températures. Relation entre les propriétés du liant et de l'enrobé.
- Section 4.2 : Influence des propriétés élastiques ou viscoélastiques des revêtements sur le comportement des ponts à dalle orthotrope.
- Section 4.3 : Formulation d'enrobés bitumineux à hautes performances grâce à l'optimisation de l'empilement granulaire et l'emploi de bitumes modifiés en polymères.
- Section 4.4 : Propriétés viscoélastiques linéaires et en fatigue des enrobés bitumineux contenant des agrégats d'enrobés recyclés.
- Section 4.5 : Comportement des matériaux composites traités au liant hydraulique et des bétons compactés routiers incluant des fraisâts bitumineux et des fibres métalliques.
- Section 4.6 : Procédés d'enrobage à température réduite: enrobés dits « semi-tièdes ».

Les sections 4.2 à 4.4 sont très liées et découlent pour partie du travail initial réalisé en thèse (section 4.1). Ainsi, les approches expérimentales et d'analyse des matériaux bitumineux, testés dans le domaine linéaire principalement, y sont très proches.

La section 4.5 présente quant à elle une démarche expérimentale et d'analyse similaire, mais appliquée à deux familles de matériaux composites (graves et bétons) traitées au liant hydraulique et incluant des fraisâts bitumineux.

La section 4.6 présente enfin les recherches liées à l'invention des enrobés dits « semi-tièdes », avec une évaluation des caractéristiques mécaniques et environnementales.

#### 4.1 Comportement thermo-mécanique des enrobés bitumineux à basses températures. Relation entre les propriétés du liant et de l'enrobé (2000-2003)

##### **4.1.1 Problématique**

La problématique de cette thèse, inscrite dans le cadre d'une collaboration entre l'ENTPE, EUROVIA, EIFFAGE et TOTAL, était de mieux appréhender les couplages thermo-mécaniques, responsables de la fissuration thermique des chaussées à basses températures. Par ailleurs, pour les laboratoires des entreprises et de l'administration, la formulation et la caractérisation du comportement des enrobés peuvent s'avérer longues et fastidieuses. Ainsi, voulait-on déterminer les relations existantes entre les propriétés des liants et des enrobés, et mieux cerner les critères existants de caractérisation des liants, voire d'en proposer de nouveaux, en relation avec le comportement en place des enrobés.

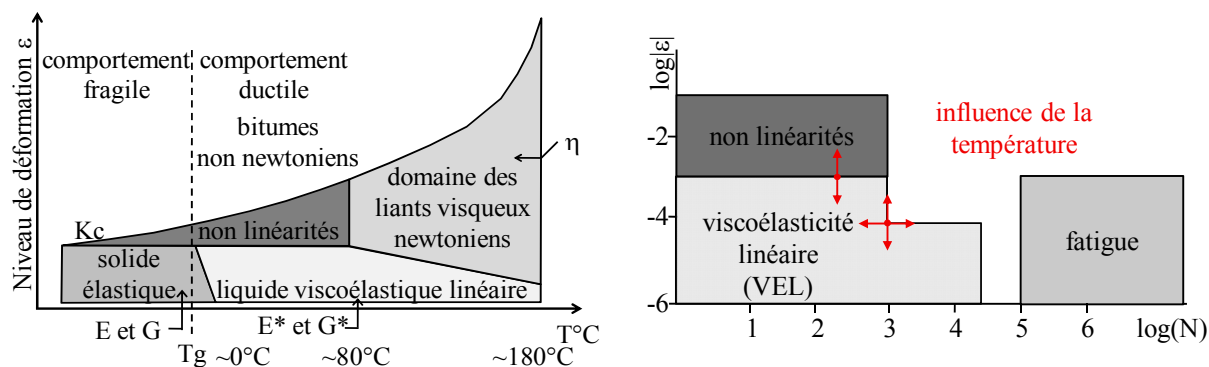
##### **4.1.2 Etat de l'art**

Les hypothèses de milieu continu et d'isotropie des liants et des enrobés sont généralement retenues. La littérature se focalise en outre sur le comportement monodimensionnel.

##### Généralités sur les propriétés rhéologiques des liants bitumineux

Les bitumes ont un comportement rhéologique complexe, dépendant de la température, de la déformation et du nombre de chargements. La Figure 1 distingue plusieurs phases de comportement :

- domaines fragile et ductile ;
- domaine élastique linéaire, caractérisé par les modules d'élasticité  $E$  et  $G$  ;
- domaine de viscoélasticité linéaire (VEL) de modules  $E^*$  et  $G^*$  ; la détermination des limites de linéarités ayant fait l'objet de nombre d'études (Peterson 1994, Airey et al. 2002, 2003, 2004) suggérant que les limites de linéarité sont fonctions de la valeur de module complexe, diminuant lorsque le module complexe augmente et, inversement, augmentant (jusqu'à des valeurs de 10%) lorsque la température augmente;
- domaine de viscosité pure, caractérisé par la viscosité  $\eta$ ;
- domaine des non-linéarités ;
- domaine de fatigue.



**Figure 1.** Classes de comportement des bitumes. (gauche) en fonction de la température  $T$  ( $T_g$  : température de transition vitreuse) et de l'amplitude de la déformation  $|\epsilon|$  [1][2] ; (droite) en fonction de  $\epsilon$  et du nombre de cycles.

### Généralités sur les propriétés rhéologiques des enrobés bitumineux

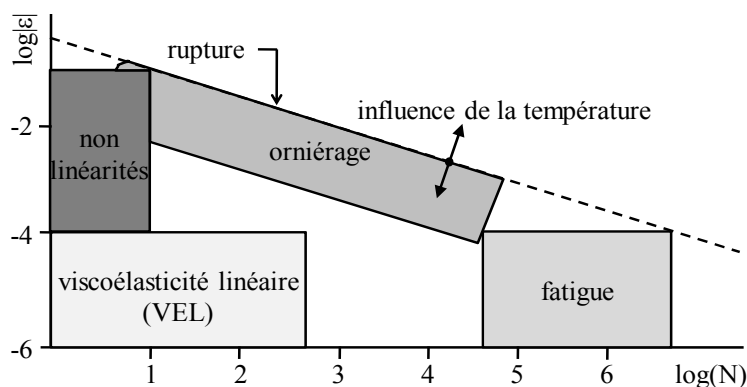
Les enrobés quant à eux présentent quatre grandes classes de comportement (Figure 2) :

- Pour un faible nombre de chargements et des déformations de quelques pour cents, le comportement observé est fortement non linéaire.
- Pour des chargements de quelques centaines de cycles et des déformations faibles ( $<10^{-4}$ ), le comportement est considéré, en première approximation, viscoélastique linéaire (VEL).
- Lors de chargements de plusieurs dizaines de milliers de cycles et des déformations faibles, les phénomènes d'endommagement apparaissent, le matériau se « fatigue ».
- En outre, lorsque des cycles déviatoires en contraintes sont appliqués à partir d'une contrainte nulle, des déformations irréversibles non négligeables existent pour des amplitudes de déformation proches de la rupture. Leur accumulation crée de l'orniérage. C'est le quatrième type de comportement qu'il convient d'identifier.

La Figure 2 permet de fixer un ordre de grandeur des différents domaines de comportement du mélange bitumineux pour une température fixée. Les frontières considérées correspondent bien évidemment à des ordres de grandeur qui peuvent varier sensiblement selon le matériau, la température et la direction du chemin de sollicitation suivi (compression, cisaillement, ...). Dans ces conditions, les quatre propriétés mécaniques importantes pour les enrobés sont :

- la rigidité ou le module (comportement viscoélastique linéaire : VEL),
- le comportement en fatigue,
- le comportement à l'orniérage (accumulation de déformations permanentes liée aux irréversibilités et aux non linéarités déjà existantes dans le domaine des faibles déformations),
- la fissuration et la propagation de fissure au sein de la structure de chaussée. En particulier, les phénomènes de fissuration liés aux variations de température (dilatation et/ou contraction thermique) constituent un aspect important qui introduit un couplage thermique/mécanique.

Charif (1991) et Doubbaneh (1995) trouvent respectivement des limites de linéarité de  $10^{-4}$  et quelques  $10^{-5}$ . Selon Airey et al., plus récemment, la limite du comportement linéaire des enrobés correspond à des déformations proches de  $10^{-4}$  et cette limite dépend peu du module complexe (donc de la température du matériau) à la différence des bitumes [Airey et al. 2002, 2003, 2004].



**Figure 2.** Comportement des enrobés, ( $\epsilon$ ) déformation – ( $N$ ) nombre de chargements [Di Benedetto 1990].

## Modélisation du comportement linéaire des matériaux bitumineux

### • Assemblage discret de ressorts et d'amortisseurs - spectre discret

Toute combinaison de ressorts (éléments élastiques) et d'amortisseurs linéaires (éléments visqueux Newtoniens) constitue un modèle viscoélastique linéaire analogique [Ferry 1980]. De nombreux modèles combinant ressorts et amortisseurs linéaires ont été présentés dans la littérature pour décrire le comportement viscoélastique linéaire des matériaux bitumineux. Les combinaisons les plus simples sont les modèles de Maxwell (liquide viscoélastique) et de Kelvin-Voigt (solide viscoélastique), représentés schématiquement en Figure 3.

Les combinaisons les plus pertinentes pour une description satisfaisante du comportement des liants et des enrobés, sont les modèles de Maxwell et de Kelvin-Voigt généralisés (Figures 4 et 5). Le modèle de Maxwell généralisé est constitué d'un groupe de  $n$  éléments de Maxwell montés en parallèle, avec éventuellement un ressort et un amortisseur linéaire en parallèle. Le modèle de Kelvin-Voigt généralisé est constitué d'un groupe de  $n$  éléments de Kelvin-Voigt en série, avec éventuellement un ressort et un amortisseur linéaire en série.

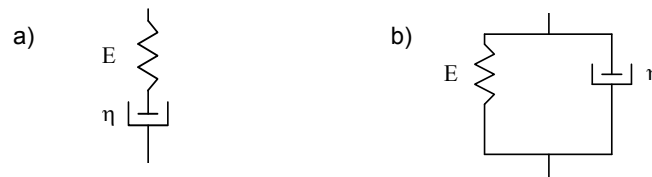


Figure 3. Eléments de Maxwell (a) et de Kelvin-Voigt (b).

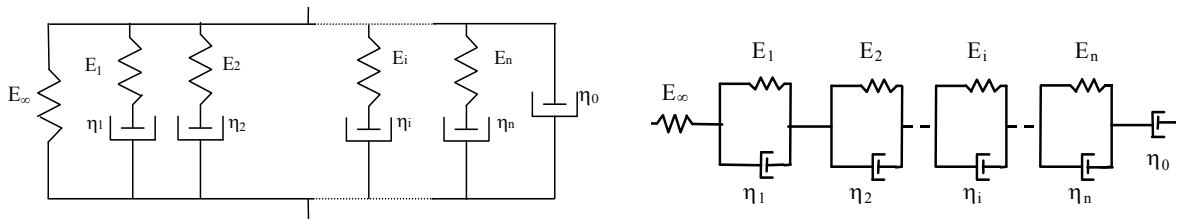


Figure 4. Modèles de Maxwell (gauche) et Kelvin-Voigt (droite) généralisés.

Un groupe d'éléments de Maxwell en parallèle représente un spectre discret de temps de relaxation, chaque temps  $\tau_i$  étant associé à une rigidité spectrale  $E_i$  ( $\tau_i = \eta_i / E_i$ ). Un ressort peut également être placé en parallèle au groupe de  $n$  éléments de Maxwell. Le module de relaxation  $R(t)$  et le module complexe correspondant  $E^*(i\omega)$  s'écrivent tels que [Ferry 1980] :

$$R(t) = E_\infty + \sum_{i=1}^n E_i e^{-\frac{t}{\tau_i}} \quad \text{Équation 1}$$

$$E^*(i\omega) = E_\infty + \sum_{i=1}^n E_i \frac{i\omega\tau_i}{1 + i\omega\tau_i} \quad \text{Équation 2}$$

Considérant, désormais, un modèle de Kelvin-Voigt généralisé (Figure 4, droite), la fonction de fluage et le module complexe sont respectivement comme suit :

$$J(t) = \sum_{j=1}^n \frac{1}{E_j} \left( 1 - e^{-\frac{t}{\tau_j}} \right) + \frac{1}{E_\infty} + \frac{1}{\eta_0} \quad \text{Équation 3}$$

$$E^*(i\omega) = \left( \sum_{j=1}^n \frac{1}{E_j + i\eta_j\omega} + \frac{1}{E_\infty} + \frac{1}{i\eta_0\omega} \right)^{-1} \quad \text{Équation 4}$$

$$\text{avec } \tau_j = \frac{\eta_j}{E_j}.$$

Plus généralement, il convient de rappeler que tout matériau visco-élastique linéaire peut être représenté par un assemblage, éventuellement avec un nombre infini, d'éléments analogiques de Maxwell ou de Kelvin-Voigt respectivement en parallèle ou en série [Mandel 1966].

- **Approches considérant un spectre continu**

Les modèles de Maxwell et de Kelvin-Voigt généralisés à  $n$  éléments ne sont pas toujours appropriés pour la description d'un comportement viscoélastique linéaire complexe. Néanmoins, en augmentant  $n$ , la modélisation du comportement est améliorée. L'utilisation d'un modèle constitué d'un nombre infini d'éléments de Maxwell ou de Kelvin-Voigt, par là même caractérisé par un spectre continu, est une amélioration possible.

Les équations 1 et 2 du modèle Maxwell généralisé mettent en exergue le spectre de relaxation discret du modèle où chaque élément a un module  $E_i$  correspondant à un temps de relaxation  $\tau_i$ . Si le nombre  $n$  d'éléments de Maxwell augmente indéfiniment, le spectre discret évolue vers une courbe continue, le module devient alors une fonction continue  $E(\tau)$  des temps de relaxation. Ainsi, la fonction de relaxation d'un tel modèle peut s'écrire comme suit :

$$R(t) = E_\infty + \int_{\ln(\tau)=-\infty}^{\ln(\tau)=+\infty} H(\tau) e^{-t/\tau} d \ln \tau \quad \text{Équation 5}$$

La fonction  $H(\tau)$  est la fonction de distribution des temps de relaxation  $\tau$ .  $H(\tau)d \ln(\tau)$  correspond au module associé au temps de relaxation  $\tau$ , dont le logarithme est compris entre  $\ln(\tau)$  et  $\ln(\tau)+d \ln(\tau)$ . Dans le domaine fréquentiel, le module complexe  $E^*(\omega, T)$  peut s'écrire :

$$E^*(\omega, T) = \int_{\ln(\tau)=-\infty}^{\ln(\tau)=+\infty} H(\tau) \frac{i\omega\tau}{1+i\omega\tau} d \ln \tau \quad \text{Équation 6}$$

Les différents modèles ayant un spectre continu peuvent être classés en deux catégories :

- les modèles dans lesquels des expressions analytiques du module complexe (domaine fréquentiel) ou des fonctions de fluage et/ou de relaxation (domaine temporel) sont proposées pour se caler aux données expérimentales,
- les modèles analogiques "avancés".

#### Expressions analytiques des fonctions visco-élastiques linéaires

Pour les bitumes, les modèles "CA" (Christensen-Anderson) et "CAM" (Christensen-Anderson-Marasteanu), introduit à la Pennsylvania State University (PSU), donnent des expressions empiriques de la norme du module complexe de cisaillement  $G^*$  et de l'angle de phase  $\delta$  [Christensen & Anderson 1992, Marasteanu & Anderson 1996, 2000] :

$$|G^*(f, T)| = \frac{G_g}{\left[1 + (f_c / f)^k\right]^{m_e / k}} \quad \text{Équation 7}$$

$$\delta = \frac{90 m_e}{1 + (f_c / f)^k} \quad \text{Équation 8}$$

Dans les deux équations précédentes,  $T$  est la température,  $f$  est la fréquence,  $f_c$  est appelé paramètre de localisation,  $G_g$  est le module de cisaillement vitreux (à très basse température et très haute fréquence) et  $k$  est un paramètre de forme sans dimension. Le modèle "CA" correspond au cas particulier où  $m_e=1$ . Dans ce cas,  $f_c$ , également appelée fréquence de croisement, correspond à la fréquence, à une température donnée, où la partie réelle et la partie imaginaire du module complexe sont égales.

Pour les enrobés, une version de la loi de puissance a été usitée dans le rapport SHRP-A-357 [Lytton et al. 1993] pour modéliser le comportement viscoélastique linéaire des enrobés. La fonction de fluage, notée par la suite  $J(t)$ , est comme suit (et pourrait être également employée pour modéliser le comportement des bitumes en première approximation) :

$$J(t) = J_0 + J_1 t^m \quad \text{Équation 9}$$

avec  $t$  : temps

$J_0$  : valeur de la fonction de fluage à  $t=0$

$J_1, m$  : constantes du matériau



Plus récemment, Zeng et al. [2001] ont proposé la généralisation suivante du modèle "CAM" valable à la fois pour les liants et les enrobés bitumineux :

$$|G^*(f, T)| = G_e + \frac{G_g - G_e}{\left[1 + (f_c / f)^k\right]^{m_e / k}} \quad \text{Équation 10}$$

$G_e$  est le module de cisaillement minimum asymptotique ( $f \rightarrow 0$ ),  $G_g$  est le module de cisaillement vitreux (module maximum asymptotique,  $f \rightarrow \infty$ ),  $f_c$  est appelé « paramètre de localisation »,  $k$  et  $m_e$  sont des paramètres de forme sans dimension. Si  $G_e=0$ , l'Équation 10 se ramène à l'Équation 7 correspondant aux modèles "CA" et "CAM". L'expression analytique donnant la courbe maîtresse de l'angle de phase est plus compliquée et n'est pas donnée ici.

#### Modèles analogiques "avancés"

Les modèles analogiques « avancés » à spectre continu (pouvant être représentés par une infinité d'éléments de Kelvin-Voigt en série ou d'éléments de Maxwell en parallèle) ont une signification et une représentation physiques. On préfère ainsi utiliser ces modèles aux expressions analytiques empiriques.

Un élément parabolique est un modèle analogique possédant une fonction de fluage de type parabolique et, en conséquence, un module complexe  $E^*$  s'écrivant comme suit :

$$J(t) = a \left( \frac{t}{\tau} \right)^h \quad \text{Équation 11}$$

$$E^*(i\omega\tau) = \frac{(i\omega\tau)^h}{a\Gamma(h+1)} \quad \text{Équation 12}$$

avec  $i$  : nombre complexe défini par  $i^2 = -1$ ,  
 $\omega = 2\pi \cdot \text{fréquence}$ , la pulsation,  
 $h$  : exposant tel que  $0 < h < 1$ ,  
 $a$  : constante sans dimension,  
 $\Gamma$  : fonction gamma<sup>4</sup>,  
 $\tau$  : temps de relaxation, dont la valeur varie uniquement avec la température.

$$\tau(T) = \tau_0 a_T \quad \text{Équation 13}$$

avec  $a_T$  : le facteur de translation. Il est égal à 1 à la température de référence choisie  $T_s$ . Une forme de  $a_T$  souvent utilisée est celle établie expérimentalement par William, Landel et Ferry (W.L.F) pour une famille de polymères [Ferry 1980], telle que :

$$\log(a_T) = \frac{-C_1(T - T_s)}{C_2 + (T - T_s)} \quad \text{Équation 14}$$

où  $C_1$  et  $C_2$  sont des constantes du matériau qui varient avec la température de référence  $T_s$ . Ce modèle analogique a un spectre continu puisque sa représentation par un modèle de Maxwell (ou de Kelvin-Voigt) généralisé nécessite une infinité d'éléments.

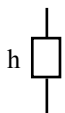
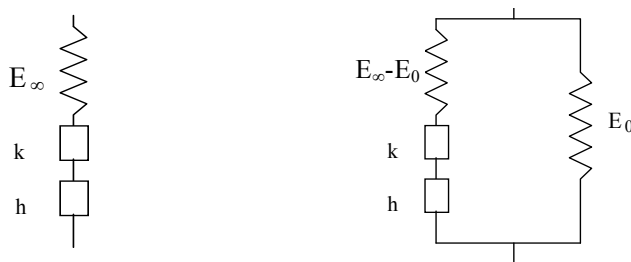


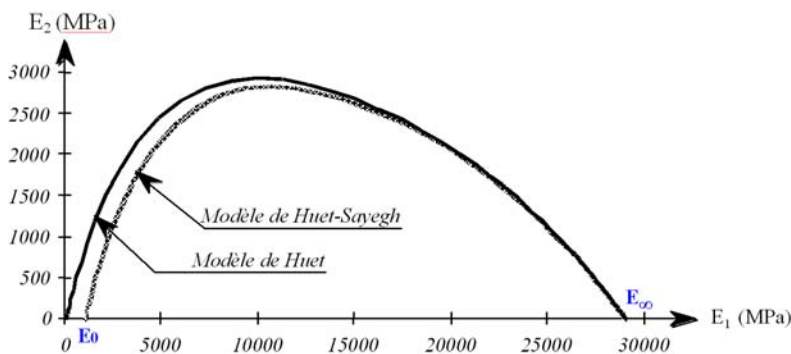
Figure 5. Représentation de l'élément parabolique.

L'élément parabolique fut utilisé par Huet (1963) et Sayegh (1965) pour développer des modèles analogiques à spectre continu adaptés au comportement VEL des enrobés (Fig. 6).

<sup>4</sup>  $\Gamma(n) = \int_0^{\infty} t^{n-1} e^{-t} dt$  avec  $n > 0$  ou  $\text{Re}(n) > 0$



**Figure 6.** Modèles de Huet (gauche) et de Huet-Sayegh (droite).



**Figure 7.** Courbes de module complexe des modèles de Huet et Huet-Sayegh dans le plan Cole-Cole.

Le module complexe du modèle de Huet-Sayegh est donné par l'expression suivante :

$$E^*(i\omega\tau) = E_0 + \frac{E_\infty - E_0}{1 + \delta(i\omega\tau)^{-k} + (i\omega\tau)^{-h}} \quad \text{Équation 15}$$

avec les mêmes notations que précédemment et :

- $E_0$  le module statique quand  $\omega\tau \rightarrow 0$ ,
- $E_\infty$  le module statique quand  $\omega\tau \rightarrow \infty$ ,
- $h, k$  : exposants tels que  $0 < k < h < 1$ ,
- $\delta$  constante sans dimension.

Seulement six constantes ( $\delta, k, h, E_\infty, E_0$  et  $\tau_0$ ) sont nécessaires pour déterminer le comportement VEL de l'enrobé à une température donnée. Ces six constantes peuvent être obtenues par un procédé d'optimisation à partir des résultats de module complexe. Si  $E_0$  est nul, les modèles de Huet et de Huet-Sayegh sont identiques.

### Modélisation du comportement non linéaire des matériaux bitumineux

Généralement, la modélisation du comportement non-linéaire des matériaux bitumineux consiste à considérer la décomposition suivante de la déformation :

$$\varepsilon = \varepsilon^e + \varepsilon^{vp} \quad \text{Équation 16}$$

avec  $\varepsilon^e$ : la composante élastique de la déformation.  
 $\varepsilon^{vp}$ : la composante viscoplastique de la déformation.

Un modèle rhéologique monodimensionnel spécifiquement développé pour les enrobés a été développé par l'équipe de Prof. Di Benedetto à l'ENTPE. Ce modèle, appelé modèle « DBN » (Di Benedetto-Neifar) introduit un comportement viscoélastique linéaire général en petites déformations, et prend en compte les non-linéarités et l'écoulement viscoplastique. Ces deux types de comportement sont reliés par une relation hyperbolique afin de simplifier la formulation. L'effet de la température est décrit aussi bien en petites déformations, où le Principe d'Équivalence Temps-Température s'applique, qu'en grandes déformations. Le modèle permet donc de décrire de manière unifiée, un comportement linéaire utilisé en pratique pour les calculs des efforts et déformations dans les structures routières, et un comportement non linéaire et irréversible (ornièrage, fissuration des chaussées, etc.) [Di Benedetto 1987, Neifar 1997, Neifar & Di Benedetto 2001] [1][2][3][8][14].



**Figure 8.** a)Modèle généralisé [Di Benedetto 1987], b)Structure choisie comme modèle discret pour les enrobés.

La description du comportement de l'enrobé, à l'aide du modèle généralisé de Di Benedetto [1987] (Figure 8a), s'effectue en qualifiant les corps non-visqueux EP<sub>j</sub> (élasto-plasticité) et visqueux V<sub>j</sub>. Le nombre de corps considérés se situe entre 10 et 20, pour trouver un compromis entre complexité des développements et description fine du comportement [Di Benedetto 1987, Neifar 1997, Neifar & Di Benedetto 2001] [1]. L'aspect discret du modèle ne doit donc pas être considéré comme une restriction puisque qu'il est possible d'étendre le modèle en considérant une infinité de corps. Le modèle décrit en Figure 8b considère des corps V<sub>i</sub> de type amortisseur linéaire, tandis que chaque corps EP<sub>j</sub> traduit un comportement proche de celui des matériaux granulaires non cohésifs. Un formalisme incrémental développé pour les sables est utilisé [Di Benedetto 1981]. Dans le cas monodimensionnel, le comportement du corps EP<sub>j</sub> est décrit par une relation entre les accroissements de contrainte (Δσ) et de déformation (Δε) à partir du dernier point d'inversion de la sollicitation :

$$\Delta\sigma = f(\Delta\varepsilon) \quad \text{Équation 17}$$

La fonction "f" de la courbe vierge a une valeur "f<sup>+</sup>" en charge avec une asymptote "s<sup>+</sup>", et une valeur "f<sup>-</sup>" en décharge avec une asymptote "s<sup>-</sup>". "s<sup>+</sup>" et "s<sup>-</sup>" sont les paliers de contrainte respectivement en compression et en traction. Ils sont déterminés à partir d'essais triaxiaux, à vitesse de déformation constante. La fonction "f" se déduit de "f<sup>+</sup>" par la relation suivante :

$$f^-(\Delta\varepsilon) = -kf^+\left(-\frac{\Delta\varepsilon}{k}\right) \quad \text{Équation 18}$$

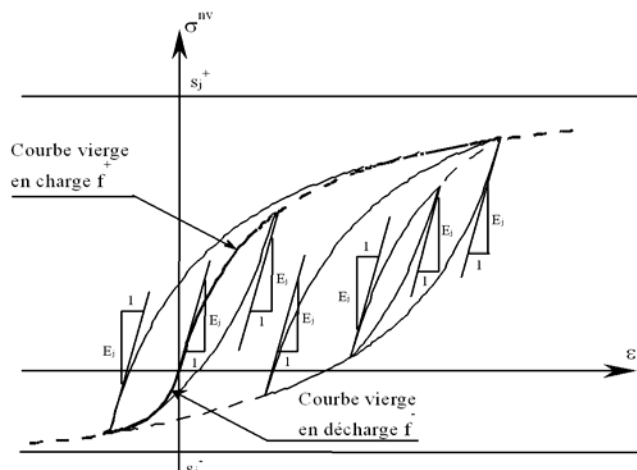
où k est le rapport entre |s<sup>-</sup>| et s<sup>+</sup>.

Dans un souci de simplicité, la fonction f<sup>+</sup> choisie est une hyperbole décrite par la pente à l'origine et l'asymptote. Seules trois constantes sont donc introduites pour chaque corps EP<sub>j</sub> :

- la pente à l'origine E<sub>j</sub>,
- l'asymptote de plasticité en compression (rupture en compression) s<sub>j</sub><sup>+</sup>,
- l'asymptote de plasticité en extension (rupture en extension) s<sub>j</sub><sup>-</sup>.

$$\sigma_j = \frac{E_j \varepsilon_j}{1 + \frac{E_j \varepsilon_j}{s_j^+}} \quad \text{Équation 19}$$

Quant à l'expression de la fonction f de décharge, il suffit de remplacer s<sub>j</sub><sup>+</sup> par s<sub>j</sub><sup>-</sup> dans l'Équation 19. La fonction f de l'Equation 17 est déduite de la fonction f<sup>+</sup> en appliquant une règle de comportement cyclique qui correspond à une extension de la règle de Masing. Quelques séquences de cyclage sont présentées Figure 9. La définition d'un critère de rupture permet de calculer s<sub>j</sub><sup>+</sup> et s<sub>j</sub><sup>-</sup> [Yan 1992, Neifar 1997]. Pour des petits cycles, le comportement est assimilable à un ressort de rigidité E<sub>j</sub>.



**Figure 9.** Comportement élasto-plastique du corps EP<sub>j</sub>; courbe vierge et exemples de séquences de cyclage.

### Influence de la composition de l'enrobé sur son comportement linéaire

Quelques chercheurs français et à l'international ont cherché à corrélérer de façon empirique le module de l'enrobé à celui du bitume, ainsi qu'à la composition granulaire et au pourcentage de vides de l'enrobé, afin de prédire un ordre de grandeur utilisable dans le dimensionnement des chaussées. En effet, la réalisation de l'essai de module complexe des enrobés est longue et coûteuse.

Les relations trouvées sont dérivées de statistiques basées sur la comparaison de résultats d'essais mécaniques sur plusieurs enrobés bitumineux ayant des compositions qui balayent les principaux paramètres de formulation. De plus, parmi les formules empiriques trouvées dans la littérature, la majorité d'entre elles concerne uniquement la norme du module complexe, l'angle de phase étant rarement étudié [Heukelom & Klomp 1964, Ugé et al. 1977, Francken & Vanelstraete 1995, 1996, 1997, 2003, Witczak et al. 1999, 2000, Zeng et al. 2001].

- **Relation de Heukelom et Klomp (1964)**

$$|E_{\text{enrobé}}^*| = S_b \left[ 1 + \frac{2.5}{n} \cdot \frac{C_v}{1 - C_v} \right]^n \quad \text{Équation 20}$$

$$\text{avec } n = 0.83 \cdot \log \frac{4 \cdot 10^4}{S_b} \quad \text{et} \quad C_v = \frac{g}{g + b}$$

où  $S_b$  (Pa) est le module du bitume tiré de l'abaque de Van der Poel [1954] donnant le module d'un bitume en fonction de la fréquence, de la température et de l'indice de pénétration (IP) du liant.  $g$  et  $b$  sont respectivement la fraction volumique des granulats et celle du bitume (%).

- **Relation d'Ugé et al. (1977)**

Cette relation donne la norme et l'angle de phase du module complexe de l'enrobé à partir des propriétés du liant et de la composition de l'enrobé.

$$A \equiv \log(E_{\infty} [\text{Pa}]) = 10.82 - 1.342 * \left( \frac{100 - g}{g + b} \right) \quad \text{Équation 21}$$

Si  $10^7 < S_b < 10^9$  Pa :

$$\log |E_{\text{enrobé}}^*| = M * (\log S_b - 8) + B \quad \text{Équation 22}$$

Si  $10^9 < S_b < 3 * 10^9$  Pa :

$$\log |E_{\text{enrobé}}^*| = B + M + (A - B - M) * \frac{(\log S_b - 9)}{\log(3)} \quad \text{Équation 23}$$

$$\text{avec : } B = 8 + 5.68 * 10^{-3} g + 2.135 * 10^{-4} g^2$$

$$M = (1.12(A - B)) / \log(30)$$

$g$  et  $b$  : respectivement la fraction volumique des granulats et celle du bitume (%)

$S_b$  : module de rigidité du bitume tiré de l'abaque de Van der Poel [1954] (Pa)

$E_{\infty}$  : module asymptotique de l'enrobé à basse température et haute fréquence

Ugé et al. (1977) fournissent également une expression empirique de l'angle de phase du module complexe de l'enrobé en fonction du module de rigidité du liant,  $S_b$ , de la teneur en vides,  $v$  (%), et de la fraction volumique du bitume,  $b$  (%). Lorsque  $S_b$  est inférieur à 2000MPa, l'angle de phase de l'enrobé est prédit en utilisant l'Equation 24 ( $S_b$  est exprimée en MPa) :

$$\varphi_{\text{enrobé}} = 16.36 * b^{0.352} * \exp \left[ 0.974 * \frac{\log(S_b / 5)}{\log(S_b / 2000)} * v^{-0.172} \right] \quad \text{Équation 24}$$

Lorsque  $S_b$  est supérieur à 2000MPa,  $\varphi_{\text{enrobé}}$  est supposé nul.

- **Relation de Francken et al. (1995)**

Tout comme Ugé (1977), Francken a montré que le module vitreux de l'enrobé à basses températures et à hautes fréquences,  $E_{\infty}$ , ne dépend que de la composition de l'enrobé:

$$E_{\infty} [\text{MPa}] = 14360 \left( \frac{g}{b} \right)^{0.55} \exp(-0.584v) \quad \text{Équation 25}$$

La norme du module complexe de l'enrobé est ensuite décomposée comme suit :

$$R^*(T, f) = E_{\text{enrobé}}^*(T, f) / E_{\infty} \quad \text{Équation 26}$$

$$\log R^* = \log B^* \left[ 1 - 1.35 \left( 1 - \exp \left( -0.13 \frac{g}{b} \right) \right) (1 + 1.11 \log B^*) \right] \quad \text{Équation 27}$$

avec :  $R^*(T, f)$ : le module réduit de l'enrobé à une température  $T$  et à une fréquence  $f$   
 $B^*$  : le module réduit du bitume, défini à partir du module en cisaillement du bitume  
 $g$  et  $b$  : respectivement la fraction volumique des granulats et celle du bitume (%)  
 $v$  : la teneur en vides de l'enrobé (%)

Francken et Vanelstraete (1995) ont également mis en évidence expérimentalement une relation entre l'angle de phase de l'enrobé et celui du liant. Cette relation, présentée Figure 9, a été vérifiée pour différents bitumes purs et modifiés.

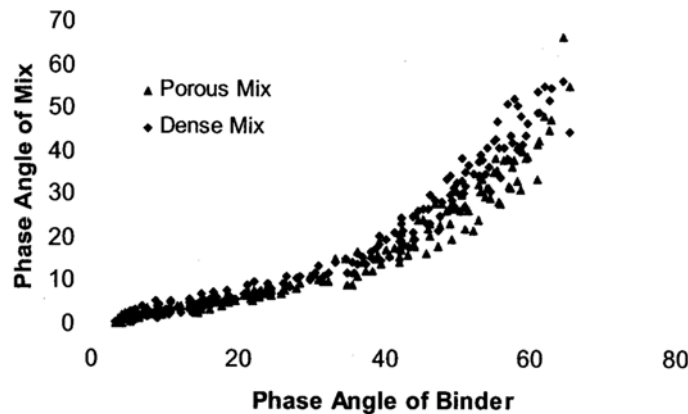


Figure 9. Relation entre l'angle de phase des enrobés et l'angle de phase des liants [Francken 1995].

- **Relation de Witczak et al. (1999)**

Plus de 2800 mesures de module complexe, sur environ 200 formulations d'enrobé différentes, ont permis d'établir la relation empirique de Witczak et al. (1999 et 2000) :

$$\log |E_{\text{enrobé}}^*| = -1.249937 + 0.029232(\rho_{200}) - 0.001767(\rho_{200})^2 - 0.002841(\rho_4) - 0.058097(V_a) - \frac{0.802208(V_{\text{beff}})}{V_{\text{beff}} + V_a} + \frac{3.871977 - 0.0021(\rho_4) + 0.003958(\rho_{38}) - 0.000017(\rho_{38})^2 + 0.005470(\rho_{34})}{1 + e^{(-0.603313 - 0.313351 * \log(f) - 0.393532 * \log(\eta))}} \quad \text{Équation 28}$$

avec :  $|E_{\text{enrobé}}^*|$  : norme du module complexe ( $10^5$ psi),  
 $\eta$  : viscosité du bitume ( $10^6$ Poise),  
 $f$  : fréquence (Hz),  
 $V_a$  : teneur en vides (%),  
 $V_{\text{beff}}$  : fraction volumique du bitume (%),  
 $\rho_{34}$  : pourcentage cumulé retenu avec une maille de 19mm (%),  
 $\rho_{38}$  : pourcentage cumulé retenu avec une maille de 9.5mm (%),  
 $\rho_4$  : pourcentage cumulé retenu avec une maille de 4.76mm (%),  
 $\rho_{200}$  : pourcentage de tamisats passant avec une maille de 75 $\mu$ m (%).

La relation suivante permet ensuite de déterminer l'angle de phase de l'enrobé:

$$\varphi_{\text{enrobé}} = -464.568 + 186.6469(\log |E_{\text{enrobé}}^*|) - 17.5259(\log |E_{\text{enrobé}}^*|)^2 + 0.326268f \quad \text{Équation 29}$$

Aussi séduisantes que puissent paraître les formules précédentes, celles-ci ont cependant été établies empiriquement sur un large échantillon de compositions d'enrobés. Such et al. ont mis en évidence que les trois premières relations, appliquées à un sable enrobé, conduisent à des erreurs importantes [1999]. Enfin, les formules précédentes ne tiennent pas compte de la taille, de la forme, ni de la rugosité des granulats définis uniquement par leur volume. Cela revient notamment à négliger le frottement interne du sable et des enrobés [Such et al. 1999].

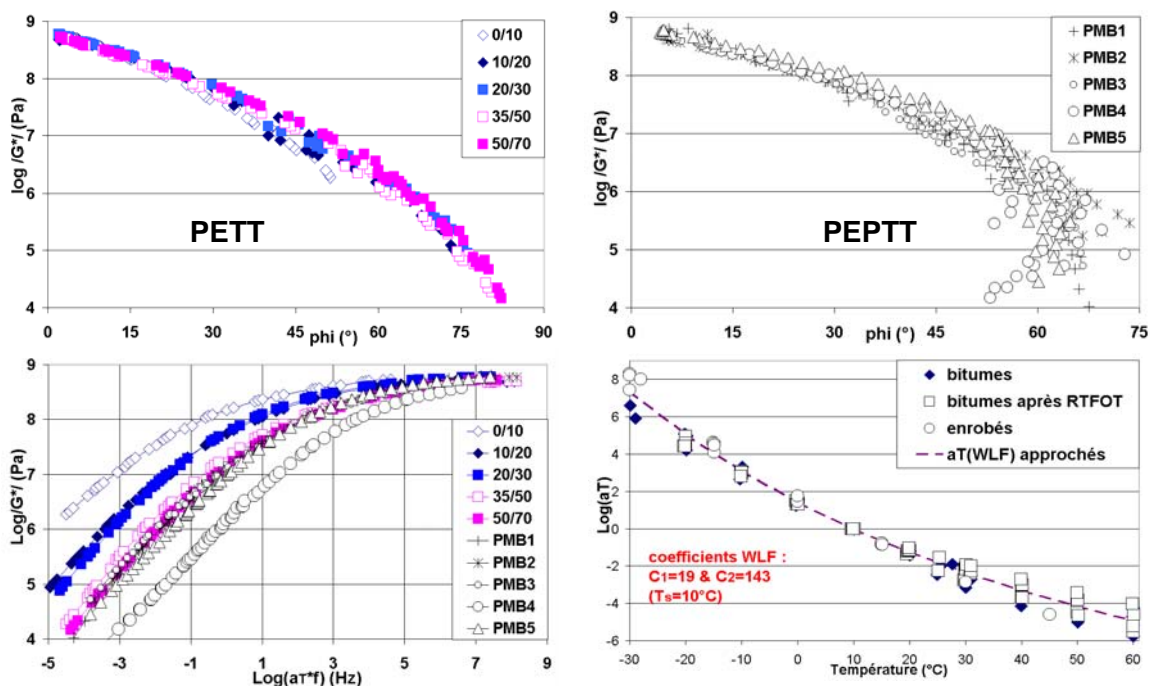
#### 4.1.3 Contributions

Au cours de ma thèse, un large travail expérimental a été mené en laboratoire sur dix bitumes différents (cinq purs et cinq modifiés en polymères) et sur cinq enrobés de même composition hormis la nature du bitume employé.

Dans le domaine des petites déformations, il ressort les points suivants:

-Les coefficients de translation  $a_T$  des bitumes et des enrobés, issus lors de la construction des courbes maîtresses de  $|E^*|$ , sont égaux sur l'intervalle de température considéré [2][5][7][9][29][30][82][85].

-Un « Principe d'Equivalence Partielle Temps-Température » (PEPTT) valable, en première approximation, pour les bitumes et les enrobés modifiés a été introduit, en se basant sur la construction de courbes maîtresses sur un seul paramètre ( $|E^*|$ , sans considérer  $\phi$ ) [2][5][9].

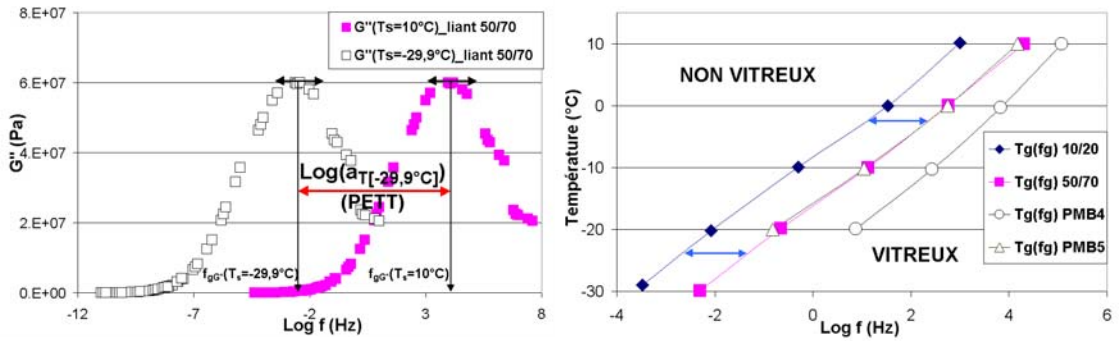


**Figure 10.** a) Module complexe des bitumes purs dans l'espace de Black ; b) Bitumes modifiés dans l'espace de Black ; c) Courbes maîtresses de  $|G^*|$  des bitumes à  $T_s=10^\circ\text{C}$  ; d) Facteurs de translation  $a_T$  de cinq liants 10/20, 50/70, PMB1, PMB4, PMB5, et des cinq enrobés correspondants, à  $T_s=10^\circ\text{C}$ .

-Une généralisation du concept de transition vitreuse a été proposée : une fréquence de transition vitreuse (notée  $f_g$ ) est introduite en se basant sur le Principe d'Equivalence Temps-Température (PETT), en particulier sur l'utilisation de la loi WLF [Ferry 1980]. Une représentation graphique d'un espace «vitreux» et d'un espace «non-vitreux» dans un diagramme d'axes  $\text{Log}(\text{fréquence})$ -température est également proposée, permettant de classer les matériaux pour toute température et toute fréquence [2][5].

$$T_g(f) = T_s - \frac{C_2 \log\left(\frac{f_g(T_s)}{f}\right)}{\log\left(\frac{f_g(T_s)}{f}\right) + C_1}$$

Équation 30



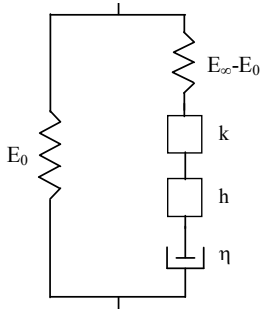
**Figure 11.** a) Détermination de la fréquence de transition vitreuse  $f_g$  (où la partie imaginaire du module complexe est maximale) à la température de référence  $T_s$ . Influence de la température de référence considérée ; b) Comportements vitreux et non-vitreux des liants 10/20, 50/70, PMB4 et PMB5 dans les axes température-Log(fréquence) pour l’essai de module complexe.

-Un modèle rhéologique monodimensionnel, qui consiste en une généralisation du modèle analogique de Huet-Sayegh, a également été développé. Ce modèle, appelé 2S2P1D, permet de simuler à la fois le comportement visco-élastique linéaire des bitumes, des mastics et des enrobés, sur une large gamme de températures et de fréquences. A une température donnée, ce modèle possède seulement 7 constantes et son module complexe est comme suit :

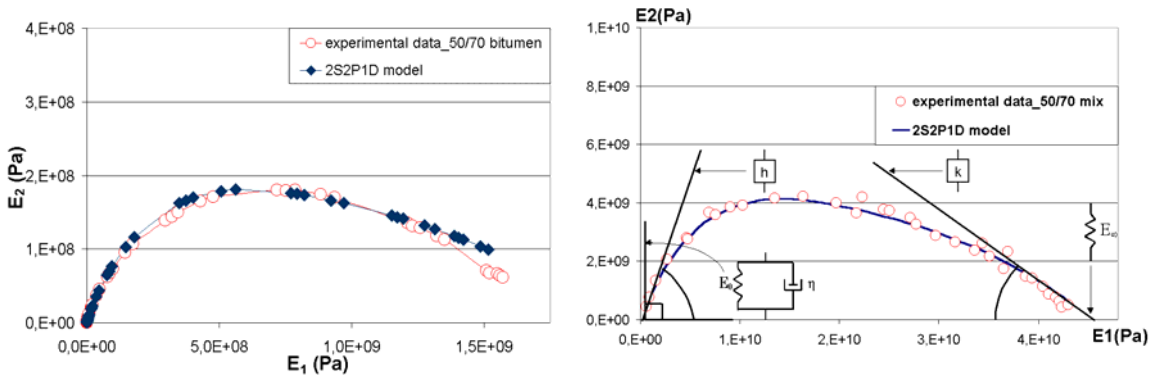
$$E^*(i\omega\tau) = E_0 + \frac{E_\infty - E_0}{1 + \delta(i\omega\tau)^{-k} + (i\omega\tau)^{-h} + (i\omega\beta\tau)^{-1}} \quad \text{Équation 31}$$

avec les mêmes notations que précédemment et l’ajout d’une viscosité Newtonienne  $\eta$  (Figure 12),  $\eta=(E_\infty-E_0)\beta\tau$ . Lorsque  $\omega\tau \rightarrow 0$ , alors  $E^*(i\omega\tau) \rightarrow E_0 + i\omega(E_\infty-E_0)\beta\tau$ .

Chaque liant et chaque enrobé correspondant possèdent les mêmes paramètres  $\delta, \kappa, \eta$  et  $\beta$ . Les seuls paramètres différents entre les modules complexes du liant et de l’enrobé sont les modules statique et vitreux ( $E_0$  et  $E_\infty$ ) et  $\tau_0$ , confirmant que la rhéologie de l’enrobé dépend directement de celle du liant pour la formulation considérée. Par identification, une fonction de transfert est proposée pour passer du module complexe du bitume à celui de l’enrobé.



**Figure 12.** Représentation du modèle général introduit 2S2P1D. L’ajout d’une viscosité Newtonienne  $\eta$  permet de simuler correctement le comportement à haute température et basse fréquence des bitumes.



**Figure 13.** Comparaison entre les résultats expérimentaux de module complexe du liant 50/70 et de l’enrobé correspondant, avec le modèle général “2S2P1D” correspondant ( $T_s=10^\circ\text{C}$ ).

-Une transformation permettant de prédire facilement et efficacement le module complexe (norme et angle de phase) de l'enrobé à partir de celui du liant est proposée. Si un bitume  $i$  et l'enrobé  $i$  correspondant –pour une formulation d'enrobé fixée– ont été testés, le module complexe d'un bitume quelconque  $j$  est alors suffisant pour prédire le module complexe de l'enrobé  $j$  correspondant, pour la formulation d'enrobé considérée. Cette formule de prédiction a été validée avec différentes formulations de mastics et d'enrobés, et revêt enfin une grande importance pratique du fait de son efficacité et sa simplicité (cf. Figure 14).

$$E_{\text{enrobé}}^*(\omega, T) = E_{0\_enrobé} + \left[ E_{\text{liant}}^*(10^\alpha \omega, T) - E_{0\_liant} \right] \frac{E_{\infty\_enrobé} - E_{0\_enrobé}}{E_{\infty\_liant} - E_{0\_liant}} \quad \text{Équation 32}$$

avec les mêmes notations que précédemment et :

$\alpha$  : coefficient dépendant de la formulation d'enrobé considérée et du vieillissement durant l'enrobage et la mise en œuvre,

$E_{\infty\_liant}$  ("module vitreux") le module du liant considéré lorsque  $\omega\tau \rightarrow \infty$  (2GPa pour les neuf liants testés dans cette étude),

$E_{0\_enrobé}$  ("module statique") le module de l'enrobé lorsque  $\omega\tau \rightarrow 0$ ,

$E_{\infty\_enrobé}$  ("module vitreux ") le module de l'enrobé lorsque  $\omega\tau \rightarrow \infty$ .

Cette relation ne repose pas nécessairement sur le PETT. De plus, même si cette relation entre les modules complexes du liant et de l'enrobé est établie dans un premier temps à l'aide du modèle 2S2P1D, le lecteur notera que cette relation est indépendante de tout modèle rhéologique. Ainsi, la relation peut directement s'appliquer aux données expérimentales brutes obtenues pour le liant considéré. Dans le Cole-Cole, l'Equation 32 a une interprétation géométrique: expansion isotrope et translation selon l'axe  $E_1$  de la courbe du liant (Fig. 15).

Trois constantes uniquement sont nécessaires pour prédire le module complexe de l'enrobé à partir de celui du liant :  $E_{0\_enrobé}$ ,  $E_{\infty\_enrobé}$  et  $\alpha$ . Les deux premières constantes possèdent une signification physique claire :  $E_{0\_enrobé}$  et  $E_{\infty\_enrobé}$  correspondent respectivement aux valeurs asymptotiques minimale et maximale de la norme du module complexe de l'enrobé, respectivement à très basses fréquences et à très hautes fréquences. Concernant le paramètre supplémentaire  $\alpha$ , il dépend de la formulation d'enrobé considérée et du vieillissement durant l'enrobage et la mise en œuvre.

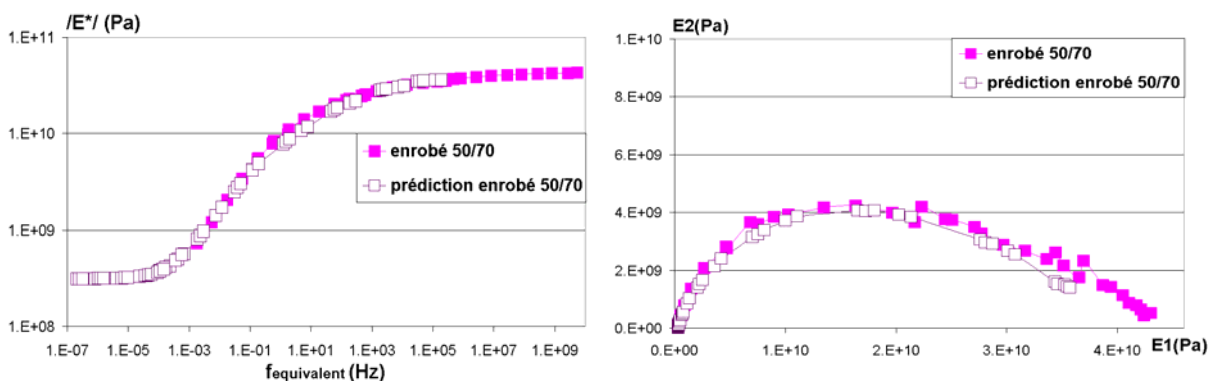


Figure 14. Comparaison entre les données expérimentales de l'enrobé au liant 50/70 et le module complexe prédit à partir de l'Equation 4 en utilisant les résultats expérimentaux du 50/70, à  $T_s=10^\circ\text{C}$ .

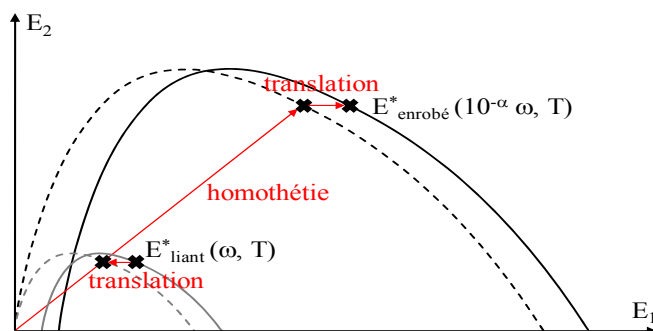


Figure 15. Prédiction du module complexe de l'enrobé à partir de celui du liant, dans le plan Cole-Cole [2][6][7].

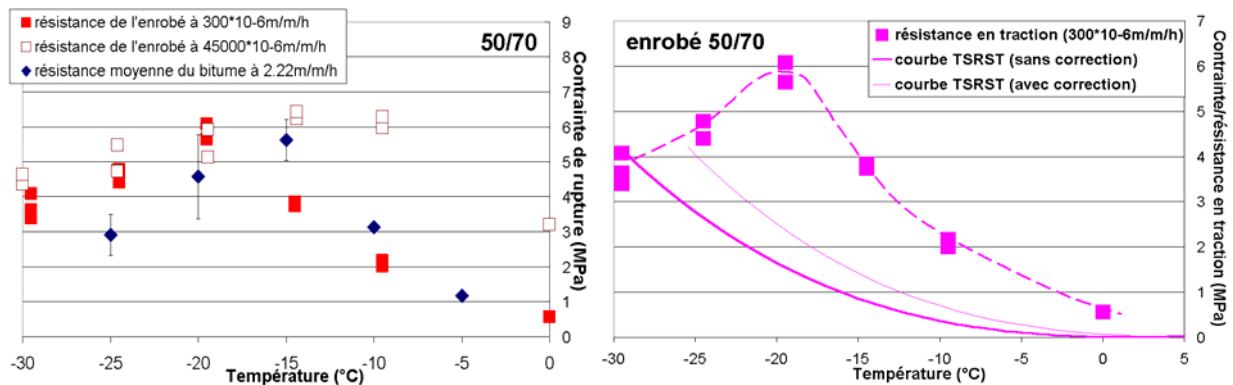


Dans le domaine des grandes déformations (comportement à la rupture), il ressort les points suivants:

-Dans le cadre d'essais de traction directe sur liants et sur enrobés à vitesse de déformation constante, la température de transition fragile/ductile correspond à la température qui maximise la résistance en traction des liants.

- A très basses températures, dans le domaine fragile, les résistances en traction des liants et des enrobés sont proches, et la vitesse de déformation n'a que très peu d'influence sur la résistance en traction des enrobés (cf. Figure 16).

-Dans le cadre d'essais de retrait thermique empêché (monotones ou cycliques), la rupture est obtenue lorsque la contrainte de traction (induite thermiquement) devient égale à la résistance en traction de l'enrobé, cette dernière pouvant être considérée comme indépendante de la vitesse de déformation dans le domaine fragile (cf. Figure 16). Une procédure de correction de la dérive thermique des extensomètres est en outre introduite.

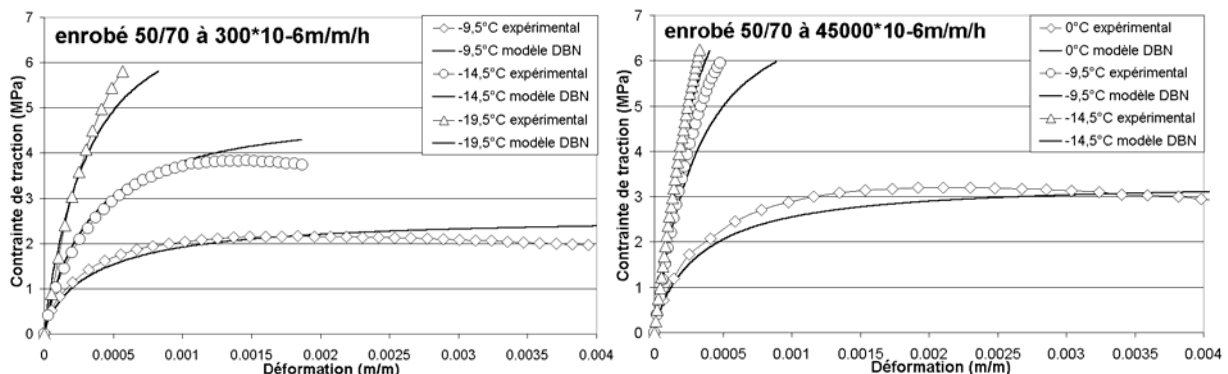


**Figure 16.** a) Comparaison des résistances obtenues aux essais de traction directe sur liant 50/70 et sur enrobé. b) Essais de traction à 300µm/m/h Vs TSRST (-10°C/h) [3][4][9][29][30][32][33][34].

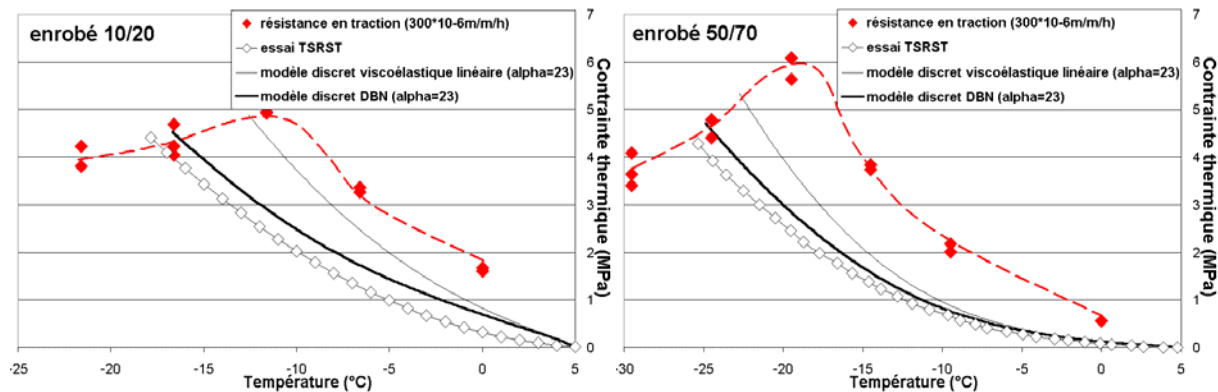
Concernant la modélisation non linéaire, en intégrant l'ensemble des points précédents correspondant aux domaines linéaire et non-linéaire, il ressort les points suivants:

-La thèse met en évidence l'efficacité et le potentiel de la loi généralisée monodimensionnelle "DBN" (Di Benedetto-Neifar) à partir de laquelle un programme informatique de simulation a été développé. Ce modèle thermo-visco-élasto-plastique permet de faire le lien entre les petites et les grandes déformations, en prenant en compte les non-linéarités du matériau.

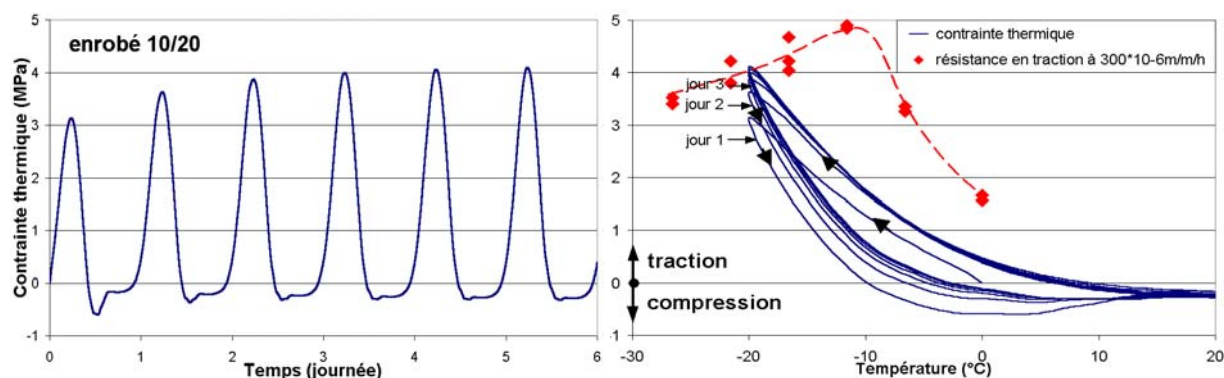
-Différentes simulations de comportement pour les cinq enrobés testés, en traction, en retrait thermique empêché monotone ou cyclique (« TSRST »), ont été réalisées. La bonne concordance entre les résultats expérimentaux et les simulations numériques présentées ont été jugés à l'époque déterminants.



**Figure 17.** Exemple de résultats expérimentaux de traction vs prédictions obtenues pour l'enrobé au bitume 50/70 (modèle DBN à 15 éléments) [2][3].



**Figure 18.** Exemples de comparaison entre la variation expérimentale de la contrainte thermique lors de retrait thermique empêché (TSRST) et différentes prédictions pour deux enrobés testés (refroidissement de  $-10^{\circ}\text{C}/\text{h}$ ) avec un coefficient de contraction thermique égal à  $23\mu\text{m}/\text{m}/^{\circ}\text{C}$  [2][3].



**Figure 19.** Calcul de la variation de la contrainte thermique pour l'enrobé au liant dur 10/20 lors d'une variation sinusoïdale de la température entre  $-20$  et  $20^{\circ}\text{C}$  [2][3].

#### 4.1.4 Discussion: positionnement du travail de thèse comparativement aux travaux ultérieurs

Le travail de thèse a permis d'appréhender et de modéliser de façon originale les propriétés thermo-visco-élasto-plastiques des liants et des enrobés bitumineux, dans un cadre monodimensionnel.

La détermination, la modélisation et l'optimisation des propriétés thermo-visco-élastiques des matériaux bitumineux constituent le fil conducteur de mes activités de recherche, depuis ma thèse jusqu'à aujourd'hui. Ainsi, les Sections 4.2 à 4.6 suivantes présentent différentes ouvertures et débouchés dans la continuité de ma thèse.

Le phénomène de fatigue, qui ne fut pas abordé pendant ma thèse, le fut quasi-systématiquement dans le cadre de mes travaux ultérieurs. Les Sections 4.2 à 4.6 abordent ainsi les propriétés en fatigue des matériaux bitumineux.

#### 4.1.5 Articles présentés en Annexe 1

Deux articles de rang A sont fournis en Annexe 1, pour illustrer l'originalité du travail de thèse :

- F.Olard, H.Di Benedetto, General "2S2P1D" model and relation between the linear viscoelastic behaviors of bituminous binders and mixes, Road Materials and Pavement Design, Volume 4, Issue 2, 2003.
- F.Olard, H.Di Benedetto, The "DBN" Model: A Thermo-Visco-Elasto-Plastic Approach for Pavement Behavior Modeling. Application to Direct Tension Test and Thermal Stress Restrained Specimen Test, Journal of the Association of Asphalt Paving Technologists (AAPT), Vol.74, pp.791-828, 2005.

## 4.2 Influence des propriétés élastiques ou viscoélastiques des revêtements sur le comportement des ponts à dalle orthotrope (2007-2011)

### 4.2.1 Enjeux techniques

Lorsque le poids propre d'un ouvrage d'art est un paramètre déterminant de son optimisation (grandes portées), un tablier à dalle métallique orthotrope constitue souvent la solution la mieux adaptée. Si le fonctionnement général de ces structures est relativement simple à comprendre, la modélisation fine de leur comportement est complexe. Leur grande déformabilité les rend en particulier très sensibles au phénomène de fatigue. Du fait des difficultés de modélisation déjà évoquées, les pathologies associées ont été évitées par l'adoption de dispositions constructives réputées sûres. Une telle démarche conduit, bien entendu, à une conception qui, bien que satisfaisante, reste assez figée.

Le comportement mécanique du revêtement pouvant être pris en compte, celui-ci devient véritablement un élément de structure, qu'il est donc naturel de chercher à optimiser, ce qui conduit à remettre en cause des dispositions constructives inadaptées aux cas non-standards. On peut alors de nouveau parler de "conception" d'une dalle orthotrope, pour laquelle les paramètres sont la géométrie des augets, leur entraxe ainsi que celui des pièces de pont, l'épaisseur du platelage, mais également la nature du matériau constitutif du revêtement ainsi que son épaisseur. On voit donc apparaître la notion de conception intégrée du couple dalle orthotrope-revêtement qui constitue la grande nouveauté du projet ORTHOPLUS.

Il s'agit, en outre, de mettre au point l'ensemble des outils (théoriques, méthodologiques et réglementaires) permettant la prise en compte du couplage platelage-revêtement, pour le calcul de la durée de vie de la structure métallique, mais aussi du revêtement lui-même. Cette dernière est en effet à ce jour mal maîtrisée, et le remplacement prématuré de l'enrobé sur un ouvrage généralement très circulé constitue un coût et une gêne à l'utilisateur non négligeables (on peut citer l'exemple récent du pont de Cheviré dans l'agglomération nantaise).

### 4.2.2 Problématique

La thèse de Simon Pouget s'inscrit dans le cadre d'une collaboration entre l'ENTPE et EIFFAGE Travaux Publics au sein du projet de recherche ORTHOPLUS, financé par l'Agence Nationale de la Recherche (ANR). L'objectif de ce projet est de développer une nouvelle conception des dalles orthotropes et de leur revêtement, depuis les études théoriques aux applications techniques, le revêtement ne devant plus se borner à un simple « poids mort » dans le cadre du dimensionnement de tels ouvrages généralement de grande portée.

Ainsi, la thèse de Simon Pouget traite de l'influence des revêtements de pont à dalle métallique orthotrope sur le comportement de l'ouvrage: d'une part, en prenant en compte le caractère visqueux des revêtements épais en matériaux bitumineux, qui constituent la solution la plus fréquente actuellement; d'autre part, en prenant en compte l'apport structurel d'un revêtement innovant en Béton Fibré Ultra-Performant (BFUP).

### 4.2.3 Contributions

Seuls sont abordés ici les mesures et les analyses numériques correspondant aux revêtements épais en matériaux bitumineux. Le cas du BFUP, élastique linéaire isotrope, n'est pas abordé.

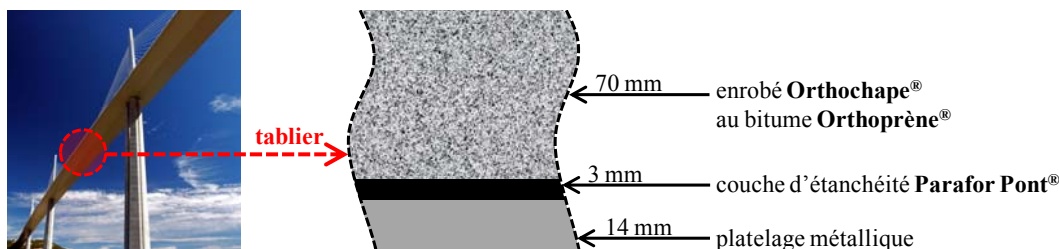


Figure 20. Composition du tablier du viaduc de Millau.

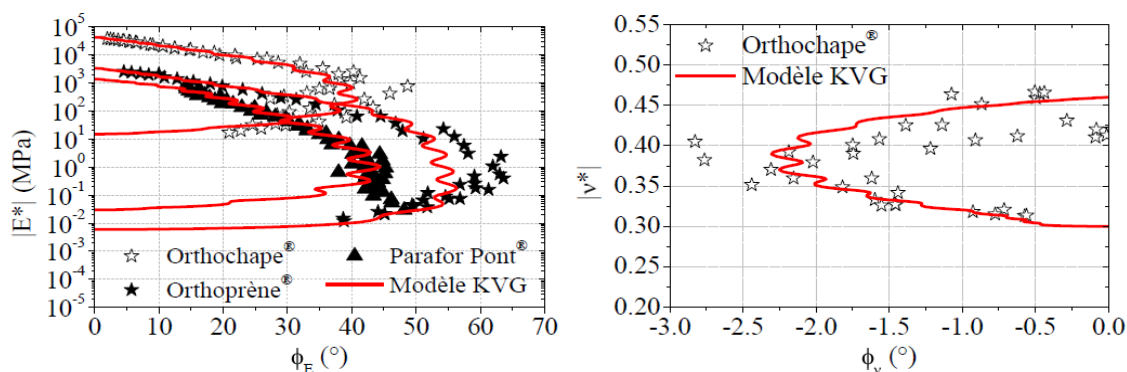
### Propriétés viscoélastiques linéaires des matériaux bitumineux du tablier du viaduc de Millau

L'essai de module complexe a été mené sur les trois matériaux considérés de la Figure 20: l'enrobé Orthochape® qui constitue la couche de roulement, le bitume Orthoprène® qui entre dans la composition de l'enrobé Orthochape®, et la couche de liaison Parafor Pont®, cf. Figures 21 et 22.

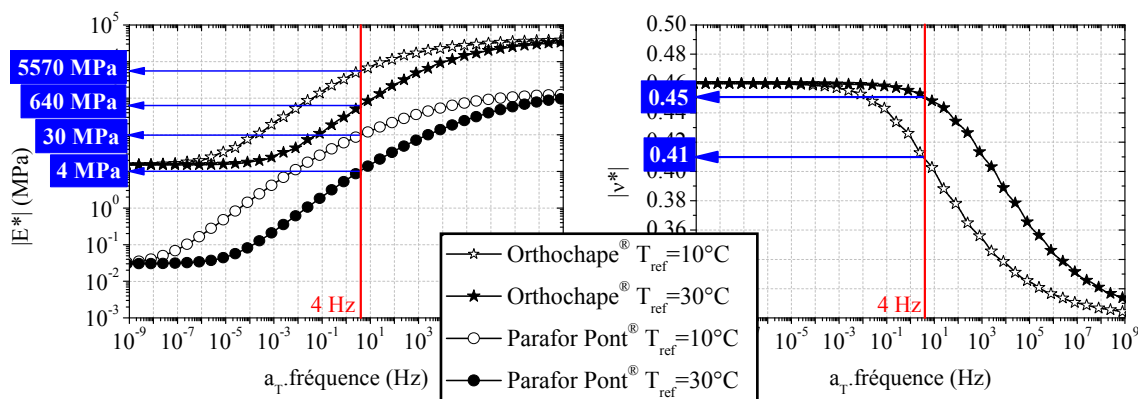
Le comportement fortement visqueux et thermo-susceptible de ces trois matériaux est mis en évidence. Pour l'enrobé, on observe un rapport de l'ordre de  $10^4$  entre les modules élevés à basses températures et/ou hautes fréquences et les faibles modules mesurés à hautes températures et/ou basses fréquences. Le coefficient de Poisson de l'enrobé, classiquement considéré constant et égal à 0.35, varie de façon importante avec la fréquence et la température ( $0.30 < |\nu^*| < 0.45$ ).

Les matériaux présentent une courbe unique dans le diagramme de Black (Figure 21), que ce soit pour le module complexe  $E^*$  que pour le coefficient de Poisson complexe  $\nu^*$  (obtenu uniquement pour l'essai de module complexe tridimensionnel sur l'enrobé Orthochape®). On peut donc dire que ces matériaux sont thermo-rhéologiquement simples et que leur comportement respecte le Principe de Superposition Temps-Température (PSTT). De plus, un même coefficient de translation  $a_T$  peut être utilisé pour les courbes maîtresses de  $(|E^*|$  et  $\phi_E)$  et pour celles de  $(|\nu^*|$  et  $\phi_\nu)$ .

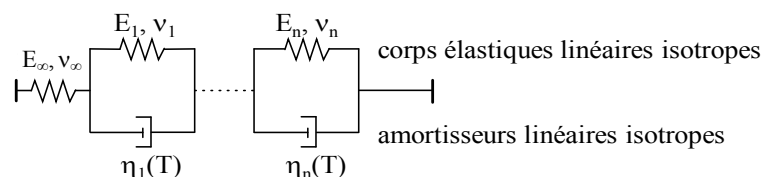
Enfin, le modèle KVG à 20 éléments simule correctement les valeurs expérimentales et le PSTT.



**Figure 21.** Module complexe  $E^*$  et coefficient de Poisson complexe  $\nu^*$  dans l'espace de Black. Simulations avec le modèle KVG (Kevin-Voigt généralisé à 20 éléments) [18][22][24][62][63][74][73][74][75].



**Figure 22.** Détermination des paramètres élastiques à partir des courbes maîtresses obtenues lors des essais sur les deux matériaux bitumineux Orthochape® et Parafor Pont® [18][22][24][62][63][74][73][74][75].



**Figure 23.** Schéma de l'expression asymptotique tridimensionnelle du modèle de Kelvin-Voigt Généralisé (KVG).

### Analyse critique de l'essai de flexion 5 points normalisé [NF P 98-286]

L'essai de flexion 5 points est décrit dans la norme NF P 98-286, le dispositif de laboratoire est décrit en Figure 24. Pour des raisons de concision, seul le cas d'un chargement appliqué sinusoïdal est ici considéré (Fig. 25). L'interprétation classique faite de ce type d'essai lors des chargements cycliques doit être révisée. En effet, la contrainte en partie supérieure du revêtement bitumineux (Fig. 26, Point A), au droit du raidisseur, alterne entre traction et compression et non en traction cyclique uniquement comme admis actuellement (Fig. 27). Ce résultat qui s'explique grâce aux effets visqueux est très important, il constitue une des contributions majeures du travail de thèse de Simon Pouget.

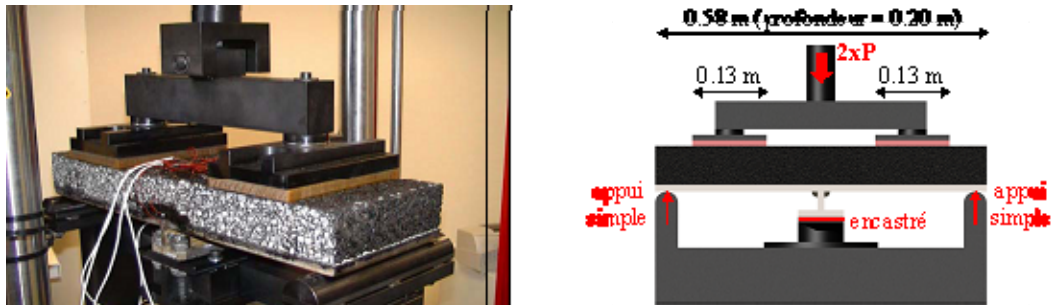


Figure 24. Dispositif de l'essai de flexion 5 points. à gauche : photo ; à droite : schéma descriptif.

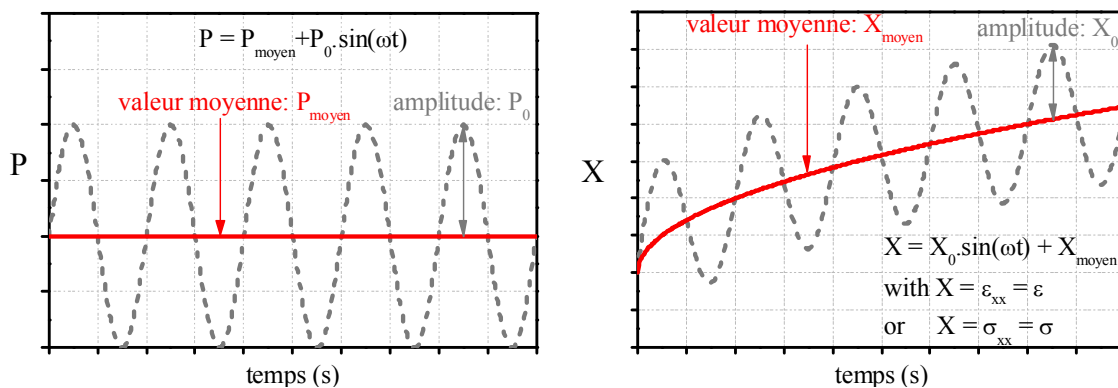


Figure 25. Représentations schématiques. gauche: du chargement P en fonction du temps; droite: d'une réponse quelconque (notée X) à ce chargement P.

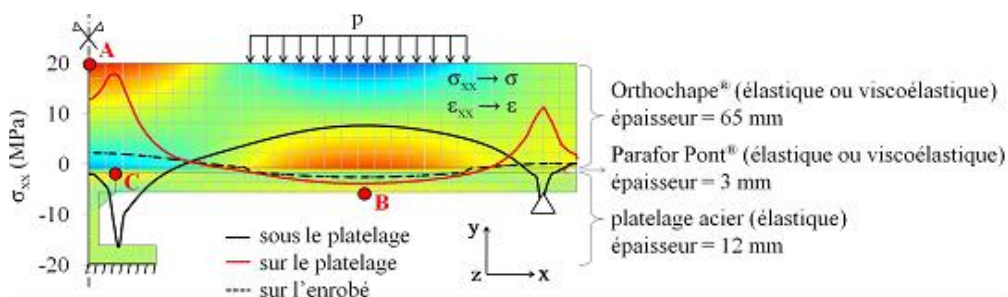
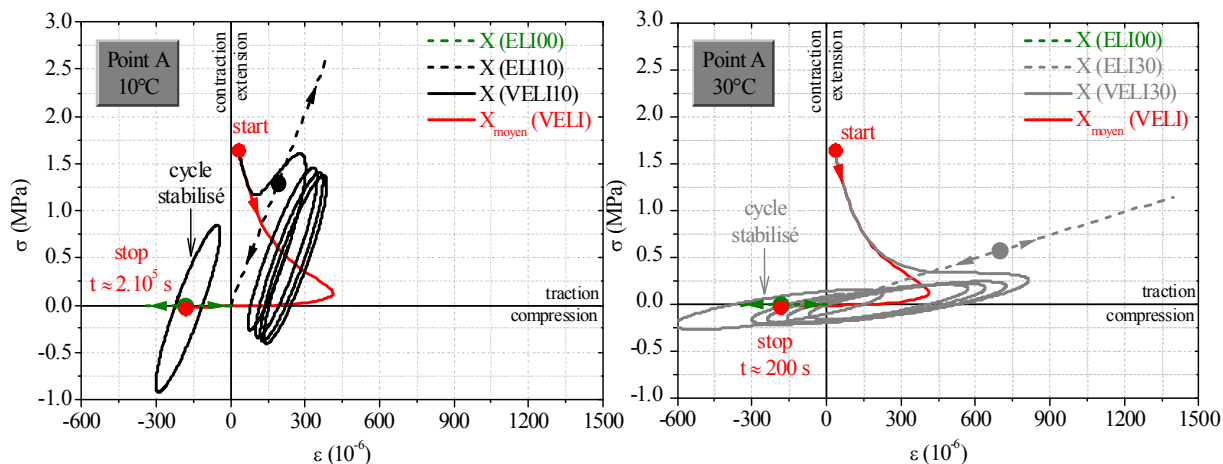


Figure 26. Maillage, conditions aux limites et champ des contraintes normales ( $\sigma_{xx}$ ) de la structure de flexion 5 points (développements éléments finis (EF) réalisés sur le logiciel COMSOL). Dans la suite en Figure 27, les résultats sont donnés au point A.



**Figure 27.** Courbes contrainte-déformation calculées au point A sur l'enrobé Orthochape®. Les calculs ELI10 et ELI130 sont réalisés à partir des valeurs déterminées en Figure 22 (à 10 et 30°C). Le calcul ELI00 repose sur l'utilisation du module minimum asymptotique  $E_0$ . Les calculs VELI10 et VELI130 considèrent un comportement viscoélastique linéaire isotrope (VELI) à 10 et 30°C, décrit par le modèle KVG à  $n = 20$  éléments [17].

### Essais sur ouvrage : le viaduc de Millau

L'originalité de la thèse de Simon Pouget, et plus généralement du programme ORTHOPLUS, tient à la complémentarité entre les investigations en laboratoire, la modélisation du comportement VELI des matériaux testés, l'instrumentation sur sites, et enfin les calculs numériques 2D et 3D.

Une instrumentation a été réalisée à l'aide de jauges bidirectionnelles donnant accès aux déformations longitudinale  $\varepsilon_{yy}$  et transversale  $\varepsilon_{xx}$  du platelage métallique. Deux lignes transversales sont instrumentées en voie lente et sous la bande d'arrêt d'urgence (sens sud-nord, Figures 28 et 29):

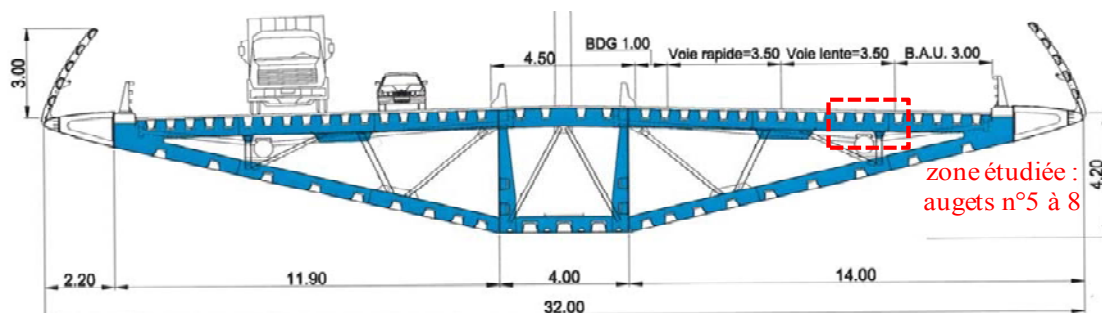
- une au droit de la pièce de pont (n°6) située à 18.45 m de la culée C0 (nord) pour laquelle des jauges sont placées en sous face de tous les augets ;
- une à mi-distance entre deux pièces de pont (n°6 et 7) à 20.53 m de C0 pour laquelle des jauges sont placées en sous face des augets et sous le platelage entre chaque auget.

S'éloigner de la zone d'appui (culée) vise à éliminer les effets de bords longitudinaux. En outre, comme le sens de circulation sud-nord a été instrumenté, il n'y a pas de problème d'effet dynamique dû au passage du camion sur le joint de chaussée. Transversalement, les augets 5 à 8 (la numérotation commence en partant du bord de l'ouvrage) sont instrumentés (Figures 28 et 29) :

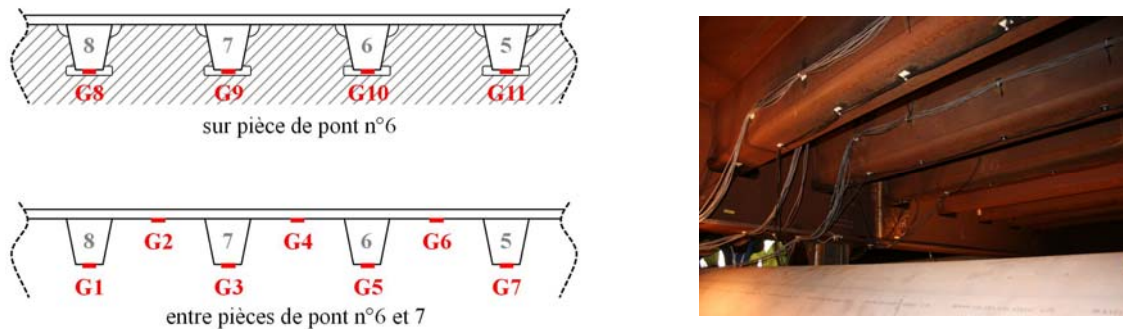
- pour faciliter la pose des jauges (hauteur du caisson à cet endroit de l'ordre de 2.40m)
- pour ne pas être trop près ni du caisson central, ni du bord du caisson latéral
- pour positionner le camion, qui sert de chargement, à cheval sur la BAU et sur la voie lente.

Deux types de chargement ont été imposés par l'intermédiaire d'un même camion :

- des chargements statiques réalisés pour différentes positions transversales et longitudinales;
- des charges roulantes, correspondant au camion lancé à deux vitesses constantes différentes (10 et 50 km/h) et pour différentes positions transversales.

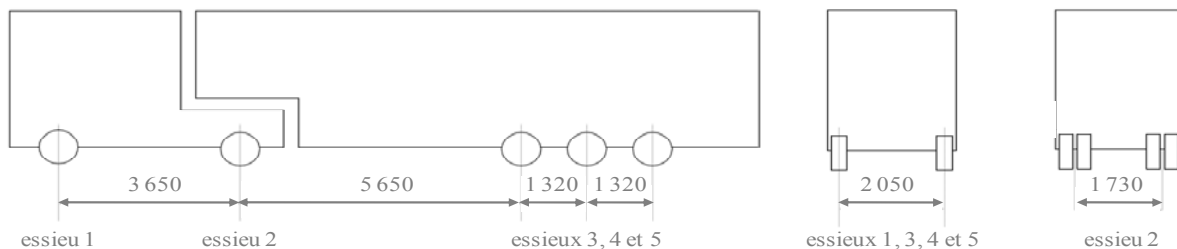


**Figure 28.** Coupe transversale du tablier au niveau de la pièce de pont n°6 [Buonomo et al. 2006].



**Figure 29.** Instrumentation mise en place sur le viaduc de Millau: positionnement et photo des jauges.

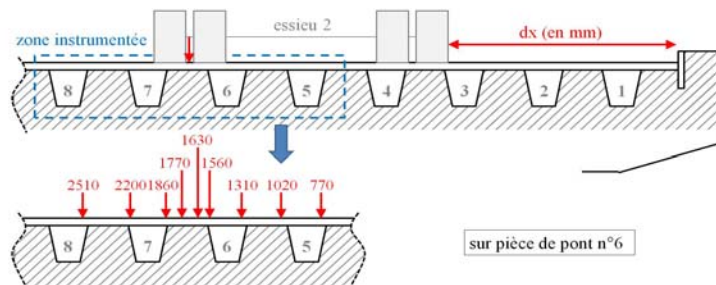
Le camion comporte cinq essieux (Figure 30). Le premier et les trois derniers essieux (essieux 1, 3, 4 et 5) sont constitués de roues simples et le deuxième essieu (essieu 2) est constitué de roues jumelées. Un pesage de chaque essieu a été réalisé pour connaître la répartition de la masse totale du camion. Le résultat des pesées est récapitulé au Tableau 1. La mesure des empreintes de chaque roue du camion a également été réalisée sur des cartons. Cela permet d'en déduire la pression appliquée par chaque roue sur le viaduc. Ces données sont précieuses, notamment pour la définition du chargement dans les calculs éléments finis.



**Figure 30.** Dimensions du camion utilisé pour le chargement du viaduc de Millau.

	Masse en tonnes
Essieu 1	6.9
Essieu 2	10.7
Essieu 3	7.1
Essieu 4	7.4
Essieu 5	7.3
<b>Total</b>	<b>39.4</b>

**Tableau 1.** Récapitulatif de la répartition, sur chaque essieu, de la masse du camion.



**Figure 31.** Schéma représentant les positions transversales  $dx$  du 2<sup>ème</sup> essieu du camion sur pièces de pont n°6.

### • Chargement statique

La température ambiante de l'air était proche de 10.5°C. Le camion a été placé sur le viaduc de manière à avoir l'essieu 2 situé à deux emplacements longitudinaux :

- à mi-distance entre deux pièces de pont (n°6 et 7) à 20.53 m de C0 noté dy1 ;
- au droit de la pièce de pont (n°6) située à 18.45 m de la culée C0 noté dy2.

Neuf positions transversales du camion ont été considérées (Figure 31), repérées par la distance  $dx$  entre le relevé d'étanchéité et le bord d'une roue de l'essieu 2 (roue droite dans le sens du camion).

### • Charge roulante

Comme pour les essais statiques, la température ambiante de l'air est proche de 10.5°C. Le camion parcourt une distance d'environ 40 mètres au dessus de la zone instrumentée à deux vitesses constantes, 10 et 50 km/h, afin de mettre en évidence l'influence du comportement visqueux du revêtement bitumineux sur la réponse de l'ouvrage. Le rapport entre les deux vitesses doit être assez important pour modifier significativement la rigidité du revêtement. Pour des raisons de sécurité, la vitesse de 50 km/h n'a pas été dépassée.

Comme précédemment, différentes positions transversales  $dx$  sont considérées. 24 essais sous charge roulante ont été menés (déroulement des essais illustré Figure 32). A titre d'exemple, les déformations de la jauge G3 sont données à la Figure 33 en fonction du temps ( $dx=1820\text{mm}$  ;  $v=48\text{km/h}$ ). Dans la suite, pour plus de clarté, seuls les maximums des déformations sont analysés. Dans l'exemple de la Figure 33, ce maximum apparaît au passage de l'essieu 4 pour la jauge G3, mais ce n'est pas nécessairement le cas pour toutes les jauges.



Figure 32. Chargement du viaduc de Millau. gauche: chargement statique; droite: charge roulante (10 et 50km/h).

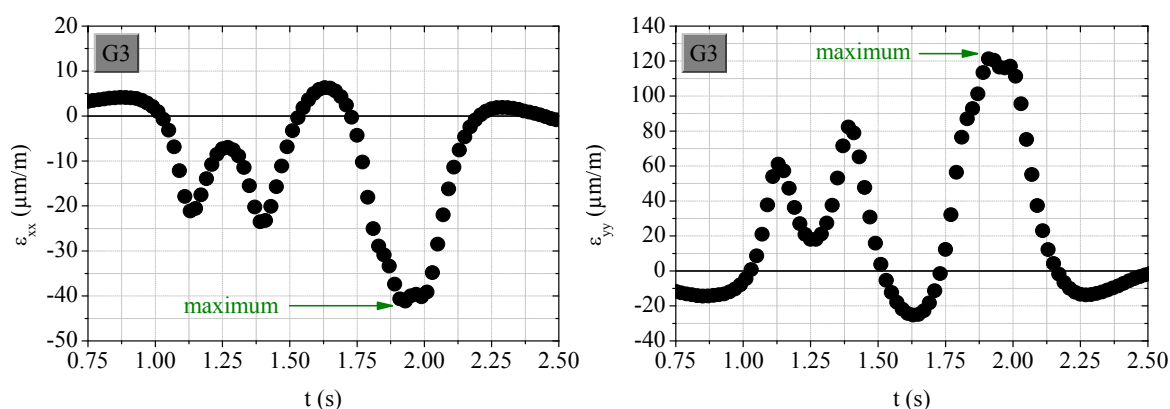


Figure 33. Exemple de mesure de la jauge G3 lors du passage n°16 (à 50 km/h)  
à gauche : déformations transversales  $\varepsilon_{xx}$  ; à droite : déformations longitudinales  $\varepsilon_{yy}$  [22][24][74][75].

### • Analyse numérique

La géométrie et le maillage EF d'un tronçon représentatif du viaduc de Millau sont illustrés Figure 34.

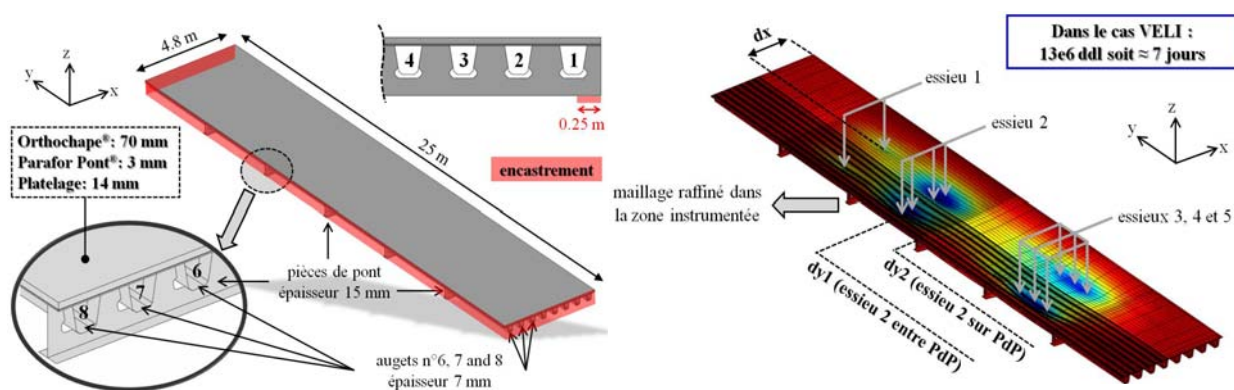


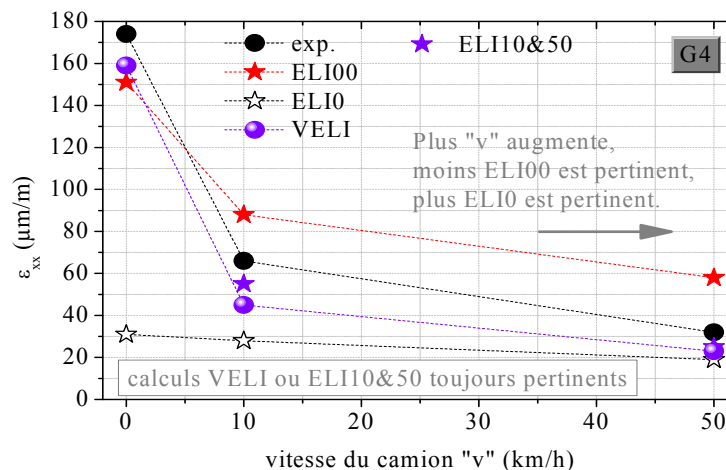
Figure 34. (gauche) Modélisation EF de la géométrie et des conditions d'appui du tronçon représentatif du viaduc de Millau ; (droite) Maillage, chargement et exemple de champ de déplacements verticaux (position  $dy_1$ , essieu 2 entre pièces de pont) [22][24][74][75].



Afin d'assurer la continuité entre le tronçon étudié et le reste de l'ouvrage, les éléments sont encastres sur trois des quatre côtés de la structure (Figure 34 gauche). En outre, pour simuler l'appui des pièces de pont sur le bord extérieur du pont, une condition d'encastrement est également considérée.

Le maillage représenté à la Figure 34 (droite) est composé de 119 664 éléments ce qui correspond à 3 120 369 degrés de liberté lorsque l'on considère un comportement élastique pour les revêtements bitumineux et à 13 117 299 degrés de liberté pour un comportement viscoélastique. Le platelage métallique, la couche d'étanchéité et l'enrobé sont maillés par des éléments volumiques quadrangles ayant une fonction d'interpolation quadratique. Les éléments de la structure métallique inférieure (augets et pièces de pont) sont eux maillés par des éléments coques triangulaires ayant une fonction d'interpolation de degré 5. La liaison entre éléments coques et les éléments volumiques du platelage métallique est un encastrement afin de simuler le plus fidèlement possible la présence de la soudure.

#### • Bilan des résultats obtenus sur le viaduc de Millau



**Figure 35.** Déformations transversales  $\varepsilon_{xx}$  mesurées et calculées dans la structure métallique par la jauge G4 à trois vitesses (0, 10 et 50 km/h) et pour  $dx \approx 1770$  mm. Nota: les calculs ELI00 et ELI0 ne donnent pas la même déformation aux trois vitesses, la position du camion étant un peu différente lors des passages correspondants.

La Figure 35, qui traduit une tendance observée sur l'ensemble des autres jauges, synthétise les résultats (expérimentaux et issus des calculs) obtenus sur le viaduc. Les calculs visqueux permettent de traduire de manière satisfaisante l'évolution des déformations dans la structure métallique quelle que soit la vitesse du véhicule. Cependant, afin de s'affranchir de ce calcul viscoélastique lourd, il est possible de réaliser, en fonction de la vitesse du camion, un calcul élastique « équivalent » :

- lorsque le camion roule, les paramètres du calcul élastique sont déterminés en utilisant les courbes maîtresses tracées à une température de référence Tref qui correspond à la température durant l'expérimentation (10.5°C), afin de déterminer la valeur du module d'Young et du coefficient de Poisson à la vitesse souhaitée.
- lorsque le camion est à l'arrêt, la méthode précédente ne peut pas être appliquée. Les paramètres élastiques équivalents correspondent alors aux valeurs asymptotiques  $E_{00}$  et  $\nu_{00}$  obtenues à basses fréquences et/ou hautes températures lors de l'essai de module complexe.

Concernant les calculs pour le cas ELI0, plus la vitesse (et donc la fréquence de sollicitation de la couche de roulement) augmente, plus ce calcul devient pertinent. Ces résultats se déduisent de l'analyse du comportement viscoélastique linéaire des matériaux bitumineux.

#### 4.2.4 Articles présentés en Annexe 2

Deux articles de rang A sont fournis en Annexe 2, pour illustrer l'originalité du travail:

- S. Pouget, C. Sauzéat, H. Di Benedetto, F. Olard, Numerical simulation of the five-point bending test designed to study bituminous wearing courses on orthotropic steel bridge, *Materials and structures*, Vol. 43, n°3, pp. 319-330, 2010.
- S. Pouget, C. Sauzéat, H. Di Benedetto, F. Olard, Effect of Vehicle Speed on the Millau Viaduct Response, *Journal of Testing and Evaluation (JOTE)*, Vol. 40, N° 7, 2012.

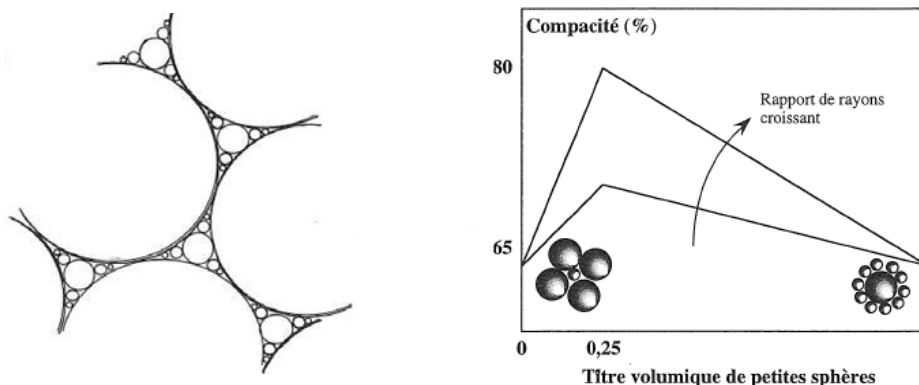
### 4.3 Formulation d'enrobés à hautes performances par un empilement granulaire optimisé et par l'emploi de bitumes modifiés en polymères (2008-2012)

#### 4.3.1 Problématique

Le problème se pose ainsi pour l'ingénieur ou le technicien routier, formulateur d'enrobés en laboratoire: étant donné les ressources disponibles en granulats (neufs ou recyclés) et en bitume, les moyens de fabrication et de mise en œuvre, les caractéristiques multiples de la chaussée à construire, comment définir et réaliser le meilleur enrobé pour une structure la plus durable possible?

Si l'on fait un peu d'histoire, alors que la formulation des bétons hydrauliques prenait un virage révolutionnaire au cours des années 1980 par la maîtrise de la floculation, la réduction de la teneur en eau et surtout l'optimisation des empilements granulaires, la formulation des enrobés bitumineux prenait un virage, certes innovant également, mais bien différent avec l'essor de nouvelles natures de bitumes dits durs, semi-soufflés, ou modifiés en polymères, l'optimisation granulaire étant ignorée (rappelons ici que les granulats représentent approximativement 95% de la masse de l'enrobé).

#### 4.3.2 Etat de l'art



**Figure 36.** Gauche: empilement "apollonien"; droite: évolution de la compacité d'un mélange binaire de sphères petites et grosses en fonction du rapport des rayons respectifs (Guyon et Troadec 1994).

Les travaux majeurs sur les arrangements granulaires optimaux remontent à Apollonius de Perga, géomètre et astronome grec (III<sup>e</sup> s. av. J.-C.) (Figure 36, gauche). Au sein d'un ensemble de sphères de même taille, Apollonios avait imaginé remplir les vides par des sphères plus petites (le diamètre du vide intérieur à un tétraèdre régulier est égal à 0,225 fois celui des sphères formant le tétraèdre) de façon séquentielle jusqu'à l'infiniment petit. Bicaux et de Gennes (1973) ont décrit les premiers la géométrie fractale de cet empilement. Le cas général d'un mélange binaire de petites sphères et de plus grosses sphères, où le ratio des diamètres respectifs de ces sphères est quelconque, est représenté en Figure 36 (droite).

Depuis, de nombreux travaux scientifiques ont été réalisés dans le domaine des bétons, dits à porosité minimale ou à hautes performances. Citons les travaux visionnaires d'Albert Caquot qui dès 1937 permit de mieux appréhender les interférences inter-granulaires (notions d'effets « de paroi » ou « de desserrement » [Caquot 1937]), sans oublier les travaux plus récents des années 80 et 90 de Baron (1982) ou encore de De Larrard (1988, 1994). Ce sujet reste d'actualité pour la communauté technique française spécialiste du béton qui est très active et prolifique sur ce sujet, intimement lié au thème clé de l'optimisation de l'emploi des ressources minérales (AFGC 2010). Plutôt que d'adopter telles quelles ces démarches théoriques et expérimentales issues du monde scientifique du béton, le parti a été pris de les adapter aux matériaux bitumineux, ainsi qu'aux outils de laboratoires routiers.

Présentons les concepts d'empilements granulaires optimaux introduits dans l'ouvrage de référence sur les bétons hydrauliques de Baron (1982). Baron schématise l'évolution de l'indice des vides d'un mélange binaire de granulats (des éléments très gros et des éléments très fins, absence d'éléments de taille intermédiaire) par trois droites (Figure 37). Deux seuils sont définis,  $p_X$  et  $p_T$ , correspondant à des concentrations critiques ( $p$  est la concentration volumique des gros granulats) qui éliminent toute interaction forte sur l'empilement des deux familles de granulats:

- Pour un mélange binaire avec une forte teneur en éléments fins,  $p < p_x$ :

$$e = F(1 - p) + Dp \quad \text{Équation 33}$$

- Pour un mélange binaire avec une forte teneur en gros éléments,  $p > p_T$ :

$$e = (C + 1)p - 1 \quad \text{Équation 34}$$

- Pour un mélange binaire intermédiaire,  $p_x < p < p_T$ :

$$e = Ep \quad \text{Équation 35}$$

où F est l'indice de vides des fins et D un coefficient lié à l'effet dit "de paroi", C est l'indice de vides des gros, et E est un coefficient sans signification physique simple.

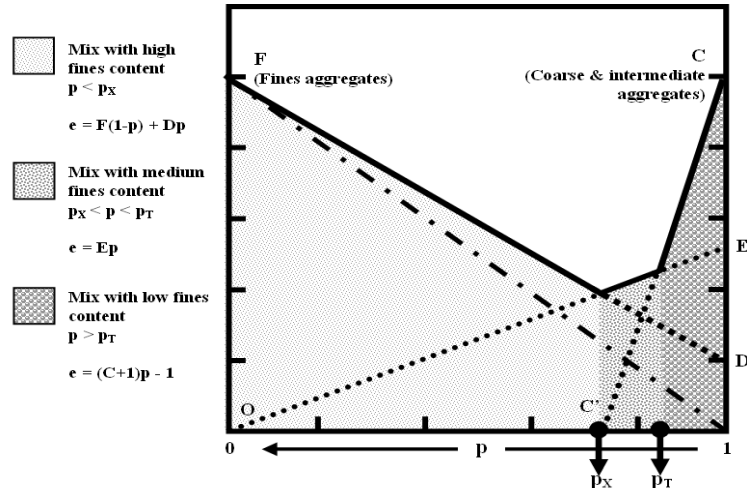


Figure 37. Variation de l'indice des vides (e) d'un mélange binaire de deux types de granulats (des fins et des gros), les fins étant considérés très fins comparés aux gros éléments (Baron 1982).

### 4.3.3 Contributions

#### Collaboration avec l'ETS (Montréal) sur l'utilisation de la Presse à Cisaillement Giratoire (PCG) pour l'optimisation des empilements granulaires [16][53]

Le principe d'optimisation granulaire de mélanges binaires de Baron a été transposé. Une procédure itérative (coupures granulaires multiples) reposant sur l'utilisation de la PCG est illustrée Figure 38. L'optimum est pris à concentration en éléments gros égale à  $p_T$ . Ce mélange binaire gros-fins est "optimal" au sens où l'indice des vides est faible (compacité élevée) tout en assurant un empilement granulaire piloté par les gros éléments. Le nombre de contacts gravillons sur gravillons est alors maximal, les éléments fins ne desserrant pas les gravillons évitant tout risque ultérieur d'ornièrage de l'enrobé. L'utilisation de la PCG à 20 girations sur granulats secs, sans bitume dans un 1<sup>er</sup> temps, s'est révélée nécessaire pour la transposition des principes présentés ci-dessus (au-delà de 20 girations, les phénomènes d'attrition, de ségrégation ou d'abrasion apparaissent).

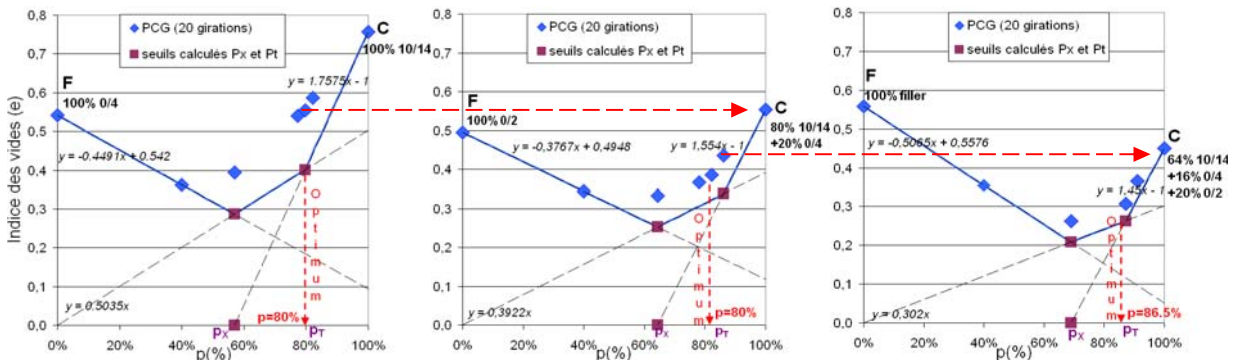


Figure 38. Optimisation itérative en 3 étapes du mélange quaternaire La Noubleau 10/14-0/4-0/2-filler, réalisée au laboratoire central EIFFAGE Travaux Publics de Corbas.

La principale contribution de ce travail collaboratif entre l'ETS Montréal et la Direction Technique d'EIFFAGE Travaux Publics est donc primordiale, dans la mesure où la méthodologie développée constitue un nouveau levier de formulation des enrobés bitumineux et d'amélioration en particulier de l'aptitude au compactage et du module de rigidité du matériau (conséquence de la densité élevée).

### Développement de la grave-bitume à hautes performances GB5®

La couche d'assise de chaussée, pour être durable, doit présenter une rigidité et une résistance en fatigue élevées; ces deux caractéristiques ont toutefois tendance à être antinomiques. Jusqu'à présent, les matériaux hydrocarbonés de la couche d'assise étaient de deux types:

- Grave bitume (GB): squelette granulaire continu enrobé à l'aide de 4 à 4,5 % de bitume pur,
- Enrobé à module élevé (EME, inventé vers 1980): la rigidité du matériau est augmentée en utilisant un bitume très dur tout en augmentant la teneur en bitume (5,5 à 6 %) pour préserver une résistance en fatigue élevée (Corté 2001, de La Roche et al. 1994). La durabilité des EME a été éprouvée sur tous les réseaux routiers en Europe (Sanders et al. 2005, de Backer 2007, Tanghe et Goos 2010), à condition que la teneur en liant demeure supérieure à 5,5% et que la densité in-situ soit supérieure à 94%. Des défauts structurels ont pu être observés sinon, se traduisant par une perte de rigidité et une fissuration prématurée (Elliott 2009).

Comparée à une formulation GB, la formulation EME engendre:

- une surconsommation en bitume (d'origine fossile) de 25 % environ,
- une fragilité à basse température et une moindre recyclabilité dues à la dureté du liant,
- un état de surface après compactage fermé et lisse, défavorable au bon engrènement avec la couche de surface (problèmes associés de viabilité hivernale).

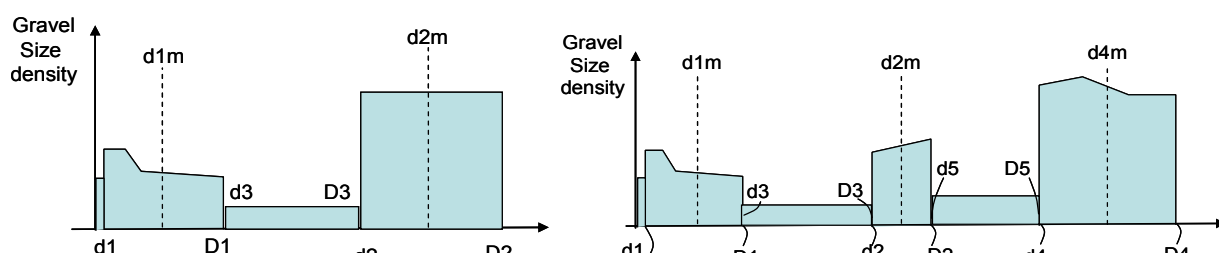
La grave bitume GB5®, fondée sur l'optimisation de la matière (granulats et bitume), répond tout particulièrement aux trois problématiques précédentes.

De manière à maximiser les contacts entre gravillons et à obtenir les densités voulues in-situ, des discontinuités granulaires ont été introduites [19][21][54][55][56][58][65][66][104][109][111][113][117]. Ont été étudiées en laboratoire, puis testées in-situ, bon nombre de granulométries (cf. Figure 39) :

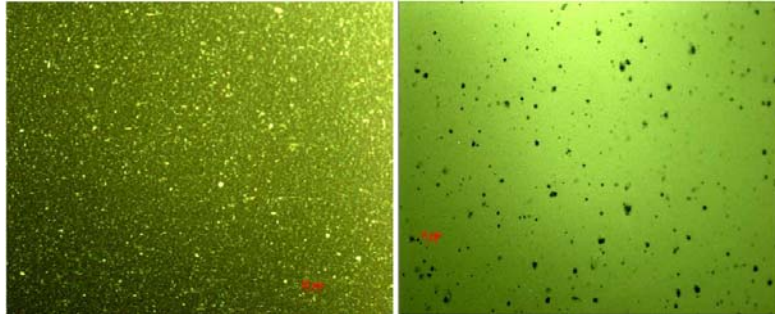
- 0/14mm avec discontinuité 4/10 ou 6/10, ou double discontinuité 2/4 et 6/10
- 0/20mm avec discontinuité 10/14 ou double discontinuité 2/4 et 10/14 ou 4/6 et 10/14

En dehors des considérations précédentes liées au squelette granulaire de l'enrobé, l'amélioration de la résistance à la fatigue et au vieillissement par modification du bitume avec des élastomères (SBS généralement) fait partie de l'état de l'art du comportement des bitumes (Baaj et al. 2005, Dressen et al. 2011). Cependant, les bitumes durs utilisés pour la formulation des EME sont généralement peu compatibles avec les SBS. C'est pourquoi, nous avons cherché à combiner à la fois la procédure précédente d'optimisation granulaire et le recours aux SBS, pour l'obtention d'enrobés à la fois très denses et rigides, mais aussi très résistants en fatigue et au phénomène de vieillissement.

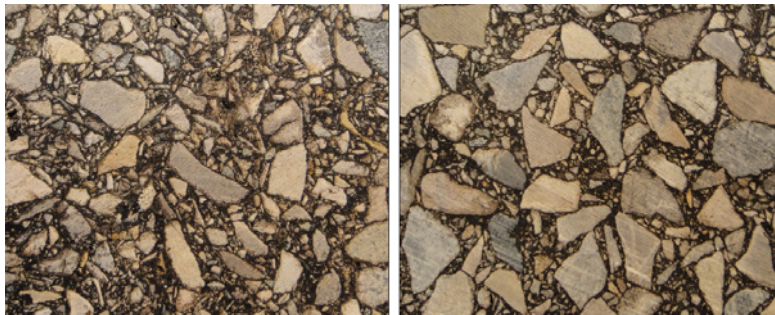
Dès lors, nous avons cherché à tester de façon très systématique en laboratoire un très grand nombre de natures et de coupures granulaires (correspondant aux carrières du groupe EIFFAGE Travaux Publics), ainsi qu'un très grand nombre de bitumes modifiés en SBS. Des bitumes faiblement modifiés (à phase continue bitume) et des bitumes fortement modifiés (à phase continue polymère) ont pu être testés (Figure 40) afin d'obtenir une palette très large de résistance en fatigue.



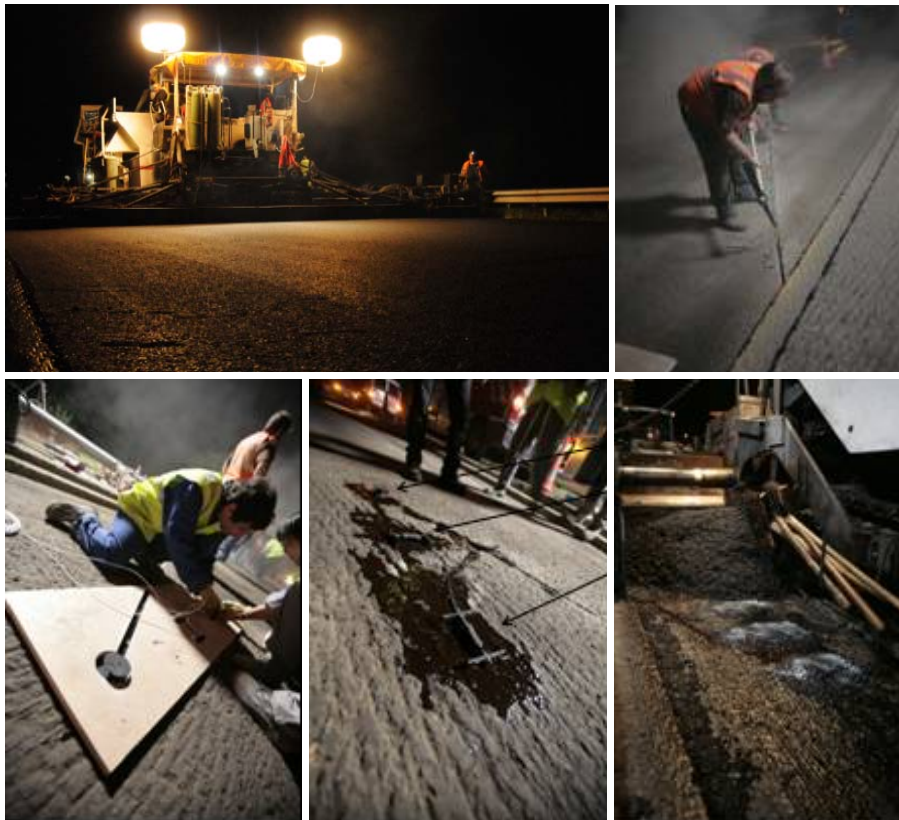
**Figure 39.** (gauche) discontinuité simple 4/10 ou 6/10 (D2=14mm) ou 10/14 (D2=20mm) ; (droite) discontinuité double 2/4 et 6/10 (D4=14mm), 2/4 (ou 4/6) et 10/14 (D4=20mm).



**Figure 40.** Microstructure de liants modifiés en microscopie de fluorescence (agrandissement x200). (gauche) liant faiblement modifié Biprène® 41R (phase continue bitume) ; (droite) liant fortement modifié Orthoprène® (phase continue polymère). [111]



**Figure 41.** Coupes d'enrobés GB3 0/14 témoin (à gauche) et GB5® 0/14 (à droite: maximisation des contacts entre gravillons), granulats Budillon-Rabatel (Izeaux).



**Figure 42.** Instrumentation d'une GB5® 0/14mm sur l'autoroute A41N (AREA). Haut droite: saignée pour le passage des câbles des capteurs. Bas gauche: mise en place des capteurs à l'aide des gabarits. Bas centre: capteurs collés à l'aide d'émulsion. Bas droite: Mise en œuvre de l'enrobé sur les capteurs. Instrumentation réalisée dans le cadre de la thèse de Philippe Gaborit (« Comportement structurel des chaussées bitumineuses épaisses du réseau autoroutier AREA », 2011-2014) co-encadrée avec l'ENTPE.

Les valeurs typiques de module et de tenue en fatigue sont présentées Tableau 2, en comparaison des spécifications actuelles des enrobés 'GB2' & 'EME2'. Un dimensionnement comparatif illustré au Tableau 3, pour un cas de trafic TC6<sub>20</sub> (trafic cumulé sur 20 ans de 6,5 à 17,5 millions d'essieux de 13 tonnes), permet de mettre en exergue les coûts relatifs et les enjeux environnementaux importants associés à ces recherches sur l'enrobé à hautes performances GB5<sup>®</sup>.

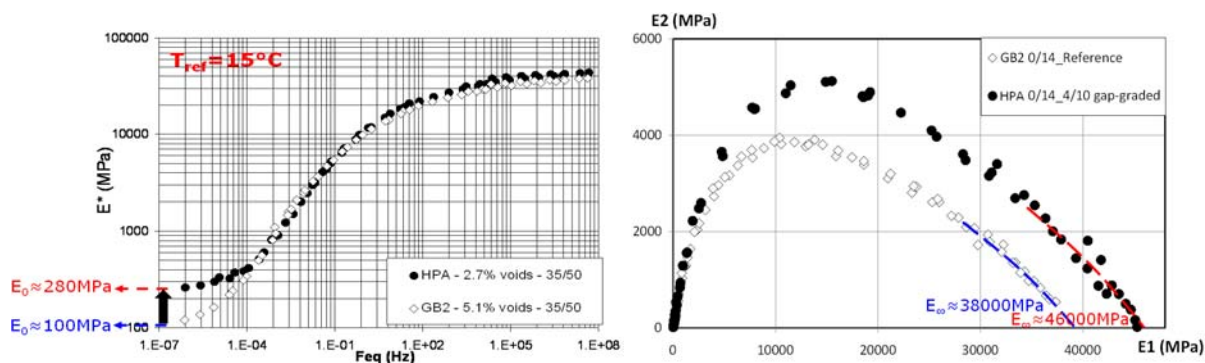
Le contexte haussier du prix du bitume et sa raréfaction encouragent la promotion d'une telle technique. Au niveau mondial, réduire de 25% la quantité de bitume au m<sup>2</sup> représente une économie annuelle potentielle de 10<sup>7</sup> tonnes de bitume (4,5 10<sup>9</sup> €) et une réduction des émissions de 3 10<sup>6</sup> tonnes de CO<sub>2</sub>. A ceci s'ajoute une moindre utilisation des classes granulaires les plus demandées en carrières (4/6, 6/10,...), d'où une optimisation possible du rendement et de la durée du gisement.

Ledit projet GB5<sup>®</sup> ayant été lauréat de l'appel à projets innovants du SETRA en 2010 (procédure nationale d'innovation routière), des sections routières et autoroutières réalisées en 2011 sont désormais suivies sur une durée de 5 ans. 250 000 tonnes de GB5<sup>®</sup> ont été réalisées à ce jour.

Caractérisation mécanique				Trafic TC6 <sub>20</sub> . Surface BBM 4cm. Plate-Forme PF3			
		E (MPa) 15°C-10Hz	ε <sub>6</sub> (10 <sup>-6</sup> ) 10°C-25Hz	Solution EME2		Solutions Innovantes GB5 <sup>®</sup>	
Spécifications "Grave Bitume 2" (GB2)		>9000	>80	Teneur en Liant=5,7%		Teneur en Liant=4%	
Spécifications "Enrobé à Module Elevé 2" (EME2)		>14000	>130	Liant 10/20 dur		Liant 35/50 + 2.5%SBS	
GB5 <sup>®</sup>	Liant 35/50 +2,5%SBS	17400 à 3,2% vides	120	Liant 35/50 + 6%SBS			
GB5 <sup>®</sup>	Liant 35/50 +6%SBS	17200 à 2,9% vides	145	4cm BBM			
Couche de surface				16cm EME2			
Couche d'assise				12cm GB5		10cm GB5	
Différence d'épaisseur				Référence		- 4cm (- 20%)	
Différence en quantité de granulats				Référence		- 20%	
Différence de quantité de bitume				Référence		- 39%	
Différence relative de coût matières /m <sup>2</sup>				Référence		-27%	
Différence relative de kg CO <sub>2</sub> eq. /m <sup>2</sup>				Référence		-17%	
						- 6cm (- 30%)	
						- 30%	
						- 48%	
						-38%	
						-28%	

**Tableau 2** (gauche) Performances des enrobés GB5<sup>®</sup> Vs spécifications GB2 & EME2. Granulats Budillon, teneur en liant 4% / masse de granulats (%vol.=10.0%).

**Tableau 3** (droite) Dimensionnement comparatif (logiciel ALIZE), coûts relatifs et impacts environnementaux.



**Figure 43.** Effet de l'optimisation granulaire ('HPA') sur les valeurs asymptotiques du module complexe  $E_0$  (à très basse fréquence et haute température) et  $E_\infty$  (à très haute fréquence et basse température). [19]

#### 4.3.4 Perspectives

Une collaboration avec l'IFSTTAR Nantes (F. Hammoum) est en cours pour évaluer la pertinence de l'analyse d'images 2D et pseudo 3D, afin d'étudier les relations entre structure interne des enrobés (au travers de quantificateurs géométriques) et performances mécaniques

A l'international, de nombreux contacts académiques sont intéressés par la technique GB5<sup>®</sup>: Associate Prof. Mihai Marasteanu (University of Minnesota), Prof. Reynaldo Roque (University of Florida), Prof. Morris de Beer (Council for Scientific and Industrial Research (South Africa)). Des licences de brevet sont en cours de commercialisation à des d'entreprises de l'hémisphère sud.

#### 4.3.5 Articles présentés en Annexe 3

Deux articles de rang A sont fournis en Annexe 3, pour illustrer l'originalité du travail:

- F. Olard, D. Perraton, On the optimization of the aggregate packing characteristics for the design of high-performance asphalt concretes, 4<sup>th</sup> EATA (European Asphalt Technology Association) Congress, Parma, Special Issue of Road Materials and Pavement Design, 2010. BEST PAPER AWARD.
- F. Olard, GB5 mix design: high-performance & cost-effective asphalt concretes by use of gap-graded curves & SBS modified bitumens, Annual meeting of AAPT (Association of Asphalt Paving Technologists), Austin (Texas), Special Issue of Road Materials and Pavement Design, Vol. 13, Supplement 1, pp. 234-259, 2012.

Le premier article est issu de la collaboration avec l'ETS Montréal. Le second présente le développement de la grave bitume GB5<sup>®</sup> à hautes performances, comme alternative technique et économique à la formulation traditionnelle des EME2.

L'extension internationale PCT WO2011FR50585A du brevet est fournie en Annexe 3.

#### 4.4 Propriétés viscoélastiques linéaires et en fatigue des enrobés bitumineux contenant des agrégats d'enrobés recyclés (2007-2012)

##### 4.4.1 Enjeux et problématique

Aujourd'hui plus que jamais, le critère d'économie de matériaux et d'économies à proprement parler est systématiquement pris en compte par les maîtres d'ouvrage, les maîtres d'œuvre et les entreprises routières. Le recyclage en centrale des enrobés bitumineux à un taux élevé est de plus en plus utilisé dans le cadre des travaux routiers et autoroutiers de réhabilitation ou de maintenance. La teneur maximale en recyclés est comprise, en général, entre 20 % et 50 % en fonction des limites matérielles des équipements de centrales considérés.

Lorsqu'à la fois la ressource en agrégats d'enrobés recyclés (AER) et la centrale d'enrobage utilisée permettent de planifier le réemploi d'un très fort taux d'AER recyclés (>50%) en couches d'assise généralement, l'étude de formulation spécifique de cet enrobé n'en demeure pas moins importante en laboratoire. Elle peut s'appuyer à bon escient sur la méthodologie des plans d'expériences factoriels et des plans de mélanges, de manière à optimiser la formulation des enrobés bitumineux contenant un fort taux d'AER.

Une problématique majeure associée à l'utilisation d'AER consiste à déterminer le degré de mobilisation du liant recyclé. En d'autres termes, le liant neuf d'apport et le liant recyclé issu de l'AER sont-ils intimement mélangés lors du process de fabrication de nos enrobés bitumineux.

##### 4.4.2 Contributions

Etudes de formulation d'enrobés à fort taux de recyclés (jusqu'à 70 %) [102][112] (2007-2010)

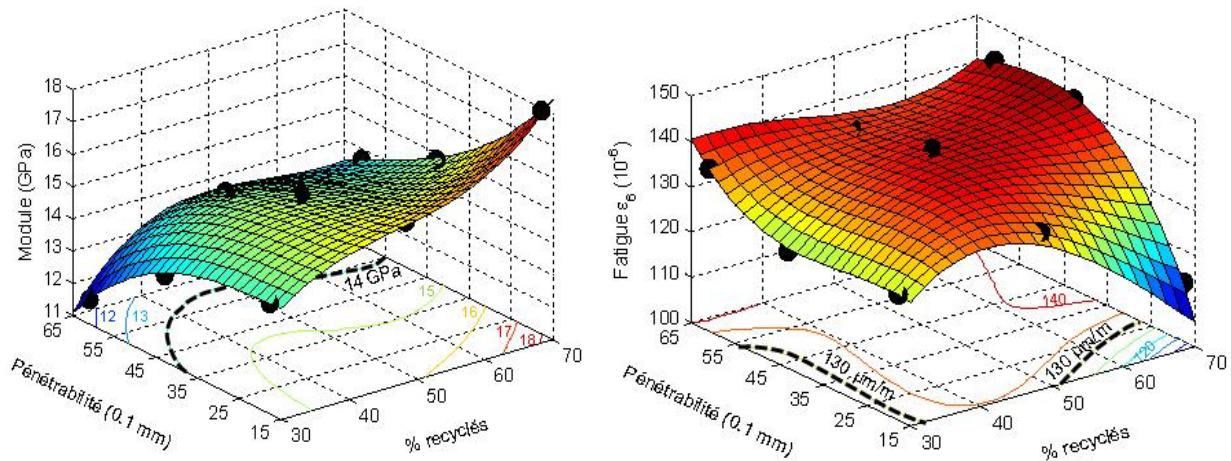
Près de 6500 tonnes d'enrobé à module élevé (EME 0/20 calcaire) à 65% d'AER furent réalisés sur l'autoroute A26 (SANEF) en 2007 et 2008. A cette occasion, une campagne expérimentale ambitieuse a été menée sur le même type de matériau selon les normes en vigueur de façon à étudier l'influence du pourcentage d'AER (3 taux de recyclage: 30%, 50% et 70%) et du grade du bitume d'apport (3 grades de consistance intermédiaire) toutes choses étant fixées par ailleurs. 9 enrobés bitumineux différents (correspondant aux 9 combinaisons possibles) ont été testés.

L'originalité des travaux consiste en ce que les essais de fatigue normalisés (EN 12697-24 2005) sur éprouvettes trapézoïdales à 10°C-25Hz ont pu être menés malgré leur caractère fastidieux –18 éprouvettes trapézoïdales testées par formule– afin d'évaluer la durabilité de tels enrobés à fort taux d'AER. En effet, de tels enrobés ne peuvent être simplement évalués sur des propriétés d'aptitude au compactage, de tenue à l'eau, de résistance à l'orniérage et de rigidité. L'utilisation d'AER à fort taux engendre usuellement une amélioration de l'ensemble de ces propriétés. C'est avant tout la notion de tenue en fatigue et donc de durabilité que nous avons souhaité évaluer.



**Figure 44.** (gauche) banc de collage des casques sur les éprouvettes trapézoïdales ; (droite) essai de fatigue à 10°C et 25 Hz, en flexion deux points sur éprouvettes trapézoïdales (EN 12697-24 2005).





**Figure 45.** (gauche) Variation du module  $E^*(15^\circ\text{C}-10\text{Hz})$  de l'EME 0/20 dans le plan (%AER, Pénétrabilité). L'isovaleur des 14 000 MPa (minimum de la spécification) est indiquée. (droite) Variation de la résistance en fatigue  $\epsilon_6(10^\circ\text{C}-25\text{ Hz})$  dans le plan (%AER, Pénétrabilité) de l'EME 0/20. L'isovaleur des 130 microdéformations (minimum de la spécification) est indiquée. [112]

Les résultats de module présentés en Figure 45 (gauche) illustrent une tendance classique: un grade dur de bitume (pénétrabilité faible) et un taux élevé d'AER engendrent un module élevé. Concernant les résultats de résistance en fatigue obtenus (Fig. 45 droite), seule la combinaison correspondant à 70% d'AER et un bitume d'apport dur (pénétrabilité 15/25) ne satisfait pas au seuil conventionnel des 130 microdéformations. Les huit autres combinaisons sont toutefois conformes, ce qui est très encourageant d'un point de vue à la fois technique, économique et environnemental. De cette observation expérimentale est née l'hypothèse d'un moindre degré de mobilisation du bitume issu du matériau recyclé (à très fort taux, à 70% plus exactement), i.e d'un moindre mélange avec le bitume neuf d'apport (en particulier dans le cas de grades durs).

Etant donné les enjeux pour la profession routière, des efforts supplémentaires restent à fournir pour poursuivre et mener à bien la reproductibilité de la démarche, notamment en l'appliquant aux formulations diverses de la profession. Sans oublier le suivi dans le temps des chantiers d'enrobés bitumineux d'assise à très fort taux de recyclés déjà réalisés.

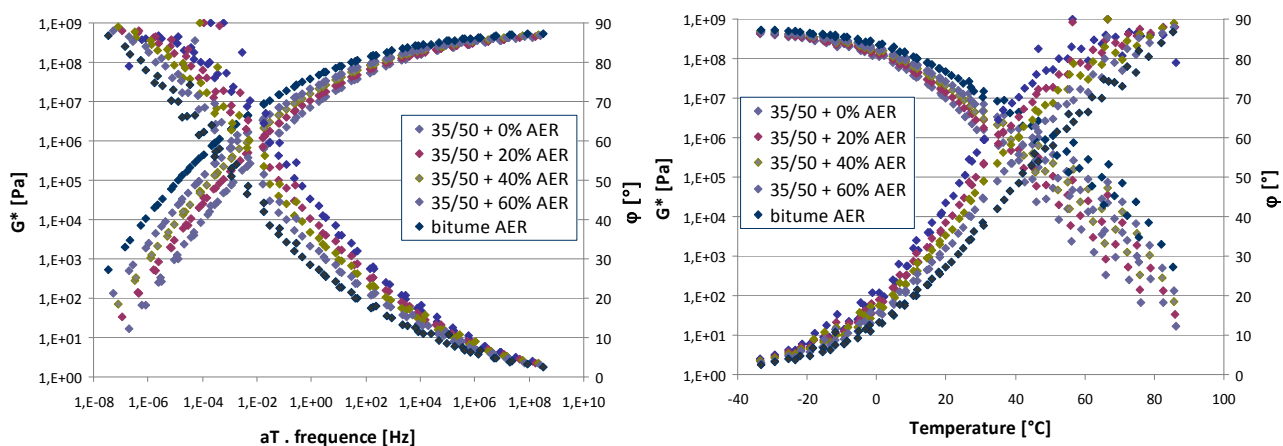
#### Degré de mobilisation du liant recyclé [26][27][79][80][81] (2010-en cours)

Une problématique majeure associée à l'utilisation d'AER consiste à déterminer une loi de mélange entre bitume neuf d'apport et bitume recyclé et indirectement à quantifier le degré de mobilisation du liant recyclé. En d'autres termes, le bitume neuf d'apport et le bitume recyclé issu de l'AER sont-ils intimement mélangés ? Si oui, peut-on le quantifier ? En effet, le cas traité précédemment en Figure 45 ne permet seulement que d'écarter le mauvais mélange constitué par un très fort taux d'ER (70%) et un bitume dur d'apport (grade 15/25), il ne permet pas de répondre à cette problématique de manière précise pour les 8 autres formules ayant été testées.

La problématique a été abordée de nouveau, mais dans le cadre d'une collaboration scientifique avec BP (France et Allemagne) et l'ENTPE. L'étude en laboratoire des propriétés viscoélastiques linéaires et à la fatigue des liants et des enrobés bitumineux avec recyclés a débuté en 2010 et fait actuellement l'objet du travail de thèse de Salvatore Mangiafico, salarié d'EIFFAGE Travaux Publics et accueilli au laboratoire universitaire du Département Génie Civil et Bâtiment (DGCB) de l'ENTPE.

La Figure 46 donne l'exemple des résultats de module complexe sur mélanges de bitume 35/50 avec différents taux de bitume extrait d'un agrégat d'enrobé recyclé (AER) fixé. L'essai de module complexe sur les enrobés correspondants ayant été par ailleurs conduit, l'objectif est ici d'utiliser le modèle 2S2P1D (cf. Équation 15) afin de mettre en exergue une loi de mélange et, dans la mesure du possible, de quantifier la qualité du mélange des bitumes dans le cas de la fabrication de l'enrobé.

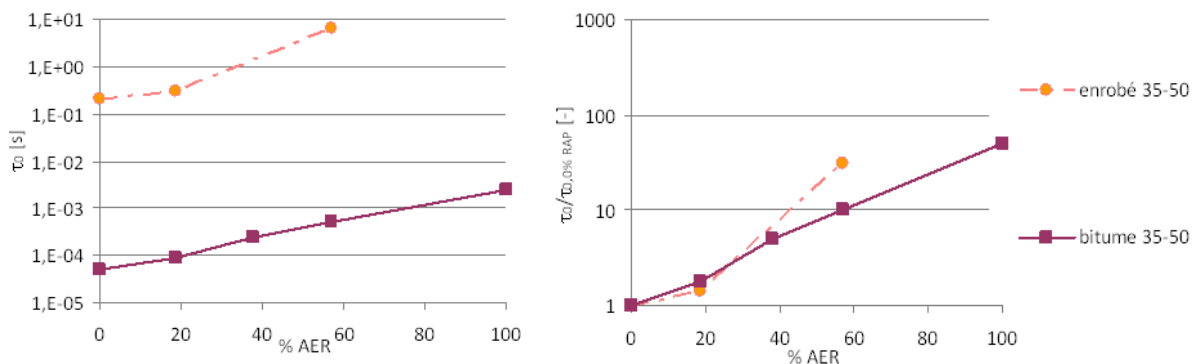
La Figure 47 donne quelque élément de réponse: les évolutions du paramètre  $\tau_0$  normalisé au-delà de 40% d'AER ne semblent plus identiques entre le bitume 35/50 et l'enrobé 35/50. L'ajout d'agent « comptabilisant » dans la composition des enrobés avec AER sera investigué pour confirmer ou non cette hypothèse d'ici la fin de thèse programmée d'ici fin 2013.



**Figure 46.** Courbes maîtresses du module et de l'angle de phase pour les mélanges de bitume 35/50 et du bitume extrait de l'AER. Gauche: isothermes à 15°C, droite: isochrones à 10Hz.

Bitumes	$E_0$ [MPa]	$E_\infty$ [MPa]	k	h	$\delta$	$\tau_0$ [s]	$\beta$
35/50	0	2450	0,191	0,539	2,85	5,00E-05	700
35/50 + 20% AER	0	2580	0,190	0,540	2,90	9,00E-05	800
35/50 + 40% AER	0	2650	0,189	0,530	3,40	2,50E-04	1000
35/50 + 60% AER	0	2740	0,188	0,528	3,25	5,00E-04	1500
Bitume issu de l'AER	0	2750	0,186	0,520	3,45	2,50E-03	1700
Enrobés	$E_0$ [MPa]	$E_\infty$ [MPa]	k	h	$\delta$	$\tau_0$ [s]	$\beta$
35/50 + 0% AER	14	36000	0,179	0,560	2,25	2,10E-01	200
35/50 + 20% AER	16	37400	0,173	0,540	2,40	3,00E-01	300
35-50 + 60% AER	20	38750	0,161	0,500	2,53	6,50E+00	900

**Tableau 4.** Paramètres du modèle 2S2P1D pour les liants et les enrobés correspondants, de 0% à 100% d'AER. Exceptés  $E_0$  et  $E_\infty$  qui dépendent de la formulation d'enrobé, et  $\tau_0$ , les autres paramètres sont jugés très proches entre les bitumes et les enrobés correspondant, confirmant ainsi indirectement l'Équation 32.



**Figure 47.** (gauche) Evolution du paramètre  $\tau_0$  en fonction de %AER (bitume 35/50 et enrobé 35/50) ; (droite) Evolution du paramètre  $\tau_0$  normalisé en fonction de %AER (bitume 35/50 et enrobé 35/50).

#### 4.4.3 Articles présentés en Annexe 4

Deux articles fournis en Annexe 4, illustrent l'originalité du travail:

- F. Olard, D. Bonneau, S. Dupriet, S. Pouget, Formulation et durabilité des enrobés à fort taux de recyclés (jusqu'à 70 %), Revue Générale des Routes et des Aéroports, N° 902, 2012.
- S. Mangiafico, H. Di Benedetto, C. Sauzéat, F. Olard, S. Pouget, L. Planque, Influence of RAP content on complex modulus of asphalt binder blends and corresponding mixes: experimental results and modeling, Annual meeting of EATA (European Asphalt Technologists Association), Braunschweig (Germany), Special Issue of Road Materials and Pavement Design, Vol. 14, 2013 [Soumission, Journal de rang A].

## 4.5 Comportement des matériaux composites traités au liant hydraulique et incluant des fraisâts bitumineux (2008-2012)

### 4.5.1 Contexte et problématique

En France comme dans bien d'autres pays, les structures de chaussées bitumineuses souples ou épaisses sont majoritaires. Les stocks de fraisâts bitumineux sont donc importants potentiellement. D'aucuns diront que la plus grande carrière dont nous disposons actuellement est la route elle-même. L'un des enjeux principaux de la profession routière est donc de redonner vie à des matériaux bitumineux fatigués, en d'autres termes c'est redonner des caractéristiques structurelles à ces matériaux en fin de vie avec peu de matériaux d'apport.

La désaffectation des maîtres d'ouvrages vis-à-vis des structures semi-rigides en raison d'une fissuration mal contrôlée et/ou de problèmes de collage à l'interface matériau bitumineux-matériau hydraulique, peut mener année après année à un déséquilibre au fil du temps et une perte des savoir-faire sur ce type de chaussées.

De plus, le trafic devenant de plus en plus agressif (nombre de véhicules, charge à l'essieu), le recours à des solutions plus résistantes et durables pourrait s'avérer nécessaire à l'avenir sur certains axes routiers, ou pour répondre à des problèmes plus locaux de renforcement (carrefour giratoire, zones de freinage ou plateforme de stockage, etc.).

Dans ce contexte, la thèse de Kevin Bilodeau a été lancée pour étudier le comportement de matériaux composites, traités au liant hydraulique et composés en partie de fraisâts ou d'agrégat d'enrobé recyclé (AER). En effet, les solutions composites granulats-bitume-liant hydraulique sont rarement utilisées et peu comprises du fait d'un comportement thermomécanique complexe.

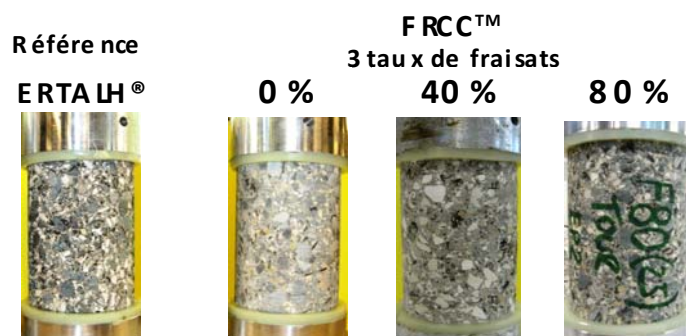
Cette thèse a été menée dans le cadre d'une convention CIFRE entre le Département de Génie Civil et Bâtiment (DGCB, CNRS UMR 5513) de l'ENTPE et EIFFAGE Travaux Publics. La thèse a été réalisée plus largement au sein du projet ANR « RECYROUTE » (Développement du béton de fraisâts compacté fibré comme matériau composite durable pour assise de chaussée à fort trafic), sélectionné suite à l'appel à propositions 2007 du Réseau Génie Civil et Urbain (RGCU). Ce projet a fédéré trois partenaires privés (EIFFAGE Travaux Publics, APRR et CTI) et quatre partenaires publics (IFSTTAR Nantes, CETE Ile-de-France, ENTPE, Ville de Paris).

### 4.5.2 Contributions

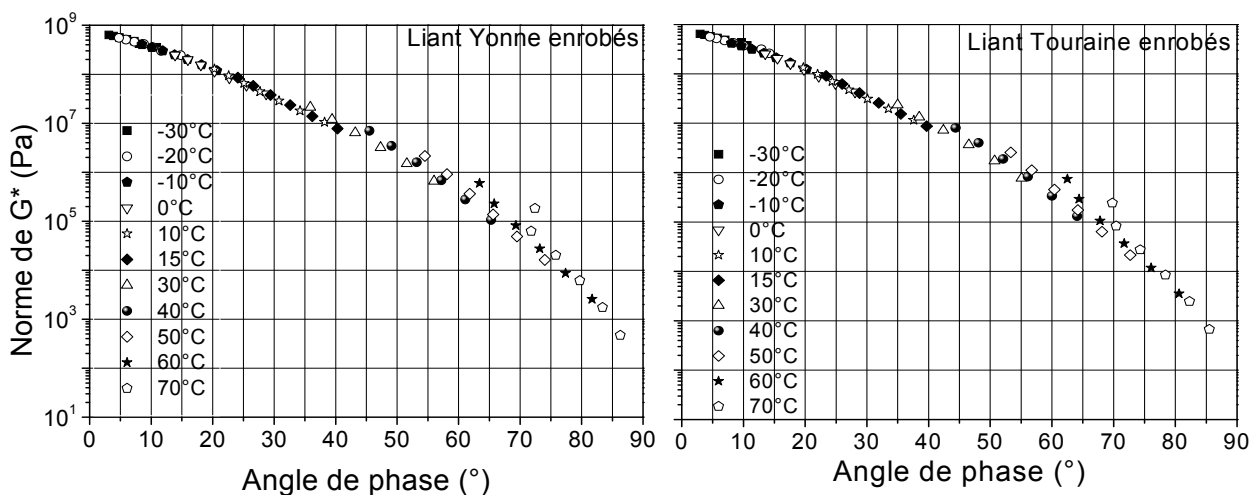
La composition des deux familles de matériaux composites testés est la suivante (Figure 48):

- le béton de fraisâts compacté fibré FRCC™, constitué de fraisâts bitumineux (différents pourcentages jusqu'à 80%), d'un correcteur granulaire, de granulats calcaires, de fibres métalliques (25 à 30kg/m<sup>3</sup>), d'un plastifiant et de 12% de liant hydraulique environ,
- la grave ERTALH® est composée de 80% de fraisâts bitumineux, d'un correcteur granulaire et de 5% de liant hydraulique environ.

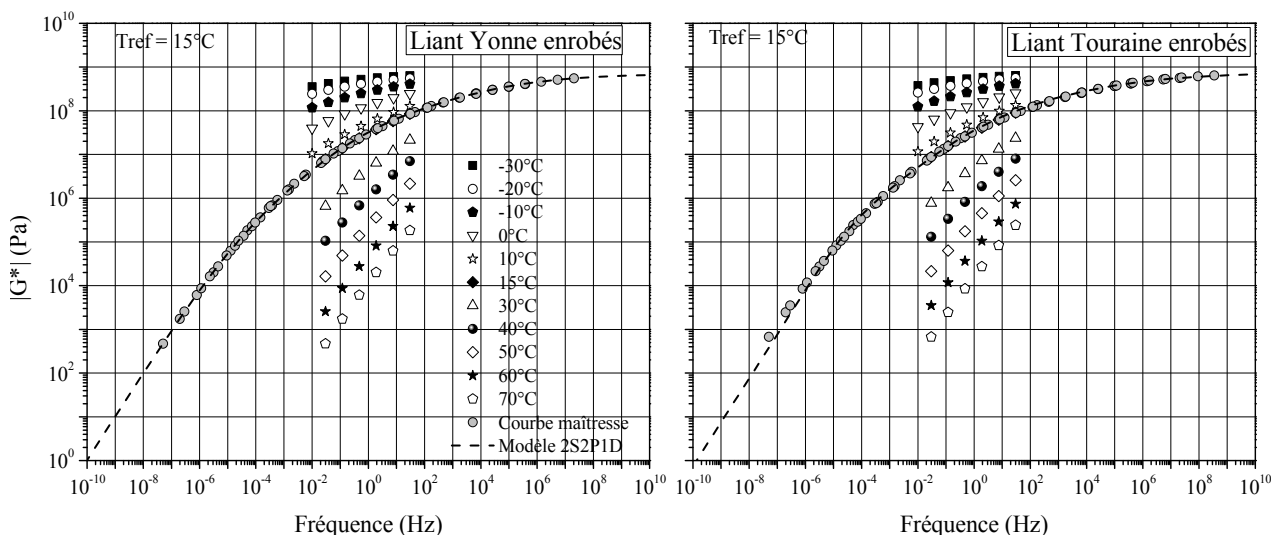
De la formulation FRCC, on cherche notamment à obtenir la réalisation d'une chaussée continue sans joint, une ouverture de fissure <1mm, un bon transfert de charge entre les bords de fissures (quasi absence de battement de dalles), et la possibilité de réduire l'épaisseur des structures de chaussées.



**Figure 48.** Exemples de matériaux composites étudiés (2 origines de fraisâts bitumineux ont été étudiées, en provenance des centrales Yonne Enrobés (89) ou Touraine Enrobés (37)).



**Figure 49.** Résultats de module complexe des liants extraits des fraisâts dans l'espace de Black: (gauche) Yonne Enrobés ; (droite) Touraine Enrobés.

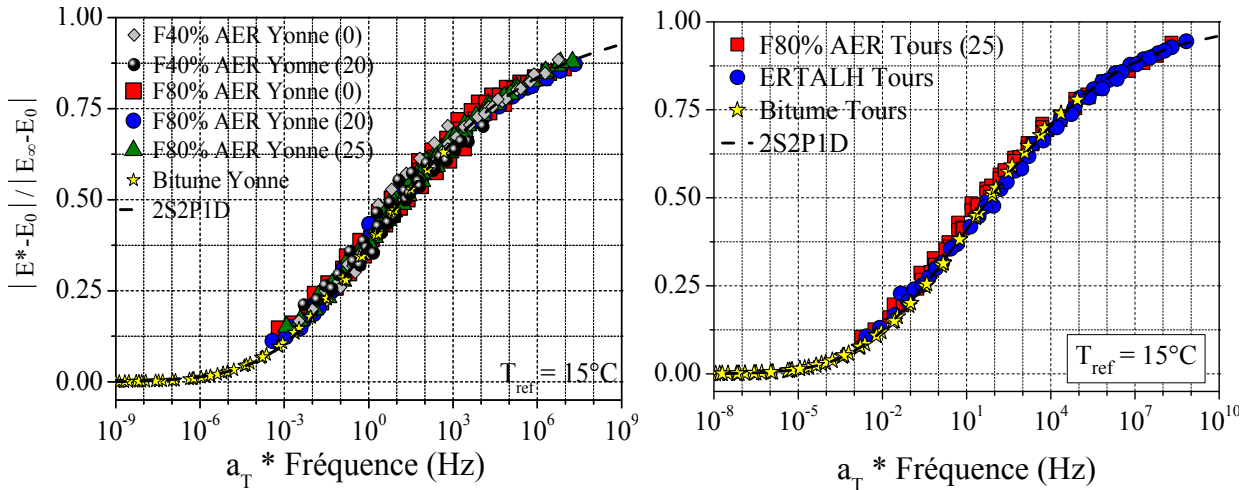


**Figure 50.** Courbe maîtresse des liants extraits des fraisâts: (gauche) Yonne Enrobés ; (droite) Touraine Enrobés.

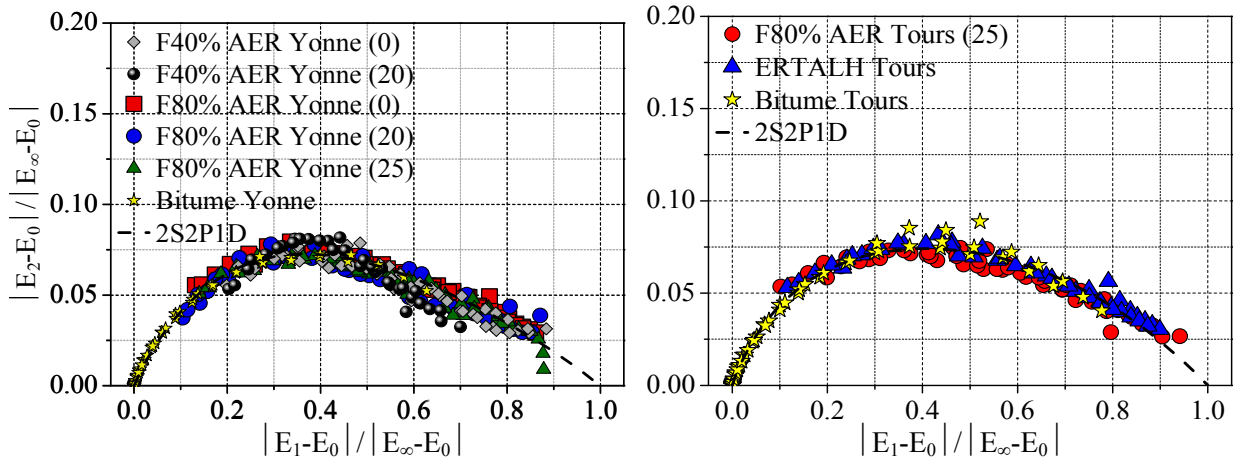
Les Figures 49 à 50 présentent les résultats de module complexe obtenus sur deux bitumes extraits de fraisâts, provenant pour l'un de la centrale Yonne Enrobés (89) et pour l'autre de Touraine Enrobés (37). Ces deux origines de fraisâts rentrent dans la composition (à hauteur de 40% ou 80%) des matériaux composites ERTALH ou FRCC testés dans le cadre de la thèse de Kevin Bilodeau. Les deux bitumes extraits présentant respectivement une courbe unique dans l'espace de Black, le principe d'équivalence temps-température (PETT) s'applique (corps thermo-rhéologiquement simples).

Les Figures 51 et 52 confortent les éléments développés pour les matériaux bitumineux, en particulier la validité de la transformation géométrique décrite par l'Equation 32. L'application de la transformation a en effet été étendue à deux types de fraisâts d'enrobés bitumineux : les fraisâts Yonne et Tours. La Figure 51 montre les courbes maîtresses du module complexe normalisé du bitume extrait des fraisâts et des matériaux composites fabriqués avec ce dernier. Sur cette figure, les courbes maîtresses du bitume et des différents matériaux composites testés se superposent. Aussi, la Figure 52 confirme la superposition des courbes du bitume extrait des fraisâts et des matériaux composites dans le plan Cole-Cole. A noter que l'utilisation du modèle rhéologique 2S2P1D, validé pour de tels matériaux composites, permet de prédire une gamme plus étendue de fréquences.

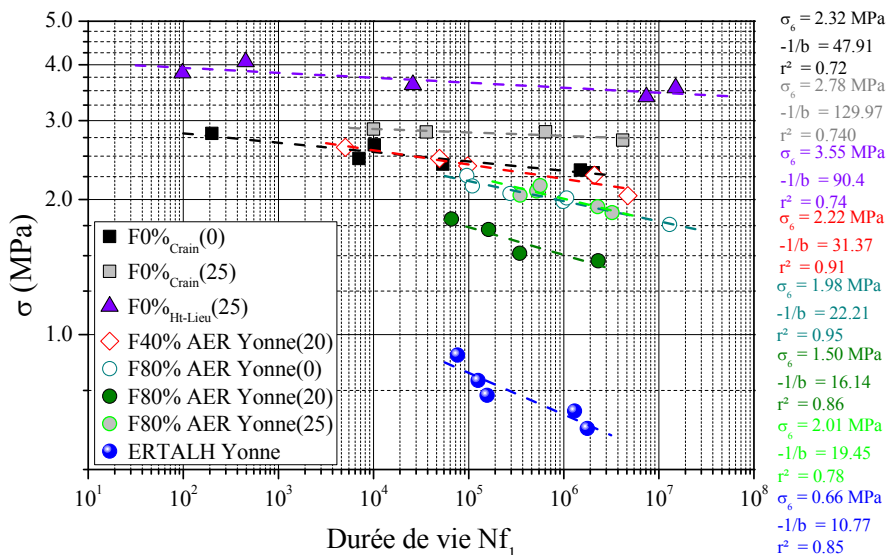
L'originalité de ces résultats tient à ce que la transformation développée initialement pour le passage bitume-enrobé peut être également un bon outil pour prédire le comportement des composites ERTALH et FRCC. Certaines propriétés visqueuses présentes dans le liant extrait des fraisâts sont en effet transmises au matériau composite, même si la matrice continue du matériau est de type hydraulique.



**Figure 51.** Courbes maîtresses normalisées pour les matériaux composites contenant les 2 types de fraisâts: a)Yonne, b)Tours.



**Figure 52.** Courbes Cole-Cole normalisées pour les matériaux composites contenant les 2 types de fraisâts: a)Yonne, b)Tours.



**Figure 53.** Droite de fatigue amplitude de contrainte  $\sigma$  en fonction de la durée de vie  $Nf_1$ .

Enfin, des essais de fatigue sur cylindres furent réalisés à 10°C-10Hz. Les courbes de Wöhler, présentées Figure 53 selon le critère usuel de perte de rigidité de 50%, montrent que la présence de fraisâts dans les matériaux FRCC diminue sensiblement les contraintes  $\sigma_6$  et les pentes de fatigue.



**Figure 54.** Mise en œuvre de la grave traitée ERTALH sur la déviation sud-est de Troyes (EIFFAGE Travaux Publics, 2008). Fabrication à partir d'une centrale mobile continue.



**Figure 55.** Machine de dosage de fibres installée sur centrale mobile (EIFFAGE Travaux Publics, 2006). Mise en œuvre de FRCC à 40% de fraisats sur le chantier expérimental de la RD 821 (28) faisant l'objet d'un suivi dans le cadre du projet ANR RECYROUTE.

Quelques exemples de réalisations récentes ERTALH et FRCC sont donnés Figures 54 et 55.

#### 4.5.3 Article présenté en Annexe 5

L'article de congrès international, fourni en Annexe 5, illustre l'originalité du travail:

- K. Bilodeau, C. Sauzéat, H. Di Benedetto, F. Olard, Roller compacted concrete for road base layer with RAP and steel fibers: viscous properties and description of experimental site, 10<sup>th</sup> International Conference on Concrete Pavements, Quebec City, 2012.

## 4.6 Procédés d'enrobage à température réduite. Evaluations mécanique et environnementale (2005-2012)

### 4.6.1 Contexte et problématique

Les enrobés à chaud sont communément fabriqués à des températures supérieures à 150°C sur des centrales continues (production continue) ou discontinues (production par gâchées successives). Les granulats sont chauffés dans un tambour rotatif pour chasser l'eau adsorbée ou absorbée dans les pores, assurant ainsi un bon enrobage du granulats et une faible sensibilité à l'eau du mélange final.

Pour des raisons économiques et environnementales, l'industrie routière a développé de nouvelles techniques d'enrobage économes en énergie et à faible émission de fumées. L'industrie a d'abord proposé les enrobés à froid, fabriqués et mis en œuvre à température ambiante [SFERB 2006, 2008], mais dont la durabilité est difficile à prévoir et liée à une variabilité élevée des mécanismes de cure et de montée en cohésion. La production annuelle européenne d'enrobés à froid en 2010 était ainsi de 5 millions de tonnes tandis que celle des enrobés à chaud s'élevait à 309 millions [EAPA 2011].

De nouveaux enrobés dits « tièdes » sont récemment apparus, permettant une réduction substantielle de la température de production (-30°C). Ces enrobés tièdes sont souvent basés sur l'utilisation de mousse de bitume ou encore d'additifs modifiant les propriétés rhéologiques du bitume pendant la fabrication et l'application du mélange. A la mise en œuvre, la température du mélange est nettement supérieure à 100°C afin d'obtenir les niveaux de compacité requis [D'Angelo et al. 2008].

Un procédé d'enrobage dit « semi-tiède », à des températures de fabrication et d'application inférieures à 100°C, est également apparu en Afrique du Sud et aux Pays-Bas, le liant étant une mousse de bitume [Jenkins 1999, 2000]. Les enrobés semi-tièdes, tout comme les enrobés tièdes présentent des propriétés mécaniques comparables à celles des enrobés à chaud.

### 4.6.2 Contributions

Mes contributions à la recherche et à l'évaluation de procédés d'enrobage à température réduite, se situent principalement dans le domaine des enrobés semi-tièdes (<100°C).

Une première contribution fut réalisée en 2005, en partenariat avec l'IFSTTAR de Nantes, dans le cadre d'un projet commun intitulé « Etude des Matériaux Traités à la Mousse de Bitume: Influence de la température et de la teneur en eau sur les propriétés mécaniques ». La bonne faisabilité de procédés d'enrobage de granulats à 20°C, 70°C et 90°C avec une mousse de bitume a été validée. Les essais de formulations classiques mettant en exergue des performances mécaniques inférieures cependant aux spécifications en vigueur pour les enrobés à chaud (l'adhésivité bitume-grulats est en particulier jugée insuffisante) [15] [84], cette piste fut abandonnée.

Une seconde contribution réalisée en interne chez EIFFAGE, débutée mi-2004 et qui perdure toujours, concerne le procédé semi-tiède dit EBT<sup>®</sup> (acronyme d'enrobés basse température) illustré en Figures 56, 57 et 58. Suite à des premiers essais de faisabilité au laboratoire central d'EIFFAGE Travaux Publics de Corbas (69), deux brevets ont été écrits, puis déposés :

- FR0503527 (08.04.2005). Extensions internationales EP1717369, US20060236614, CA2541622, ZA200602861, NZ546419, AU2006201444, MA28260, AR56965, BR200601348. [115]
- FR0507655 (08.04.2005). Extensions internationales EP1712680, US20060240185, CA2541625, ZA200602862, NZ546420, AU2006201443, MA28261, AR55907, BR200601349. [116]

Des travaux ont été menés à bien au sein des laboratoires d'EIFFAGE Travaux Publics et de l'IFSTTAR principalement [37][38][40][41][42][44][46][47][48][49][51][52][59][92][94][95][98][99][101][102][105], dans les laboratoires de l'ENTPE et de l'ETS dans une moindre mesure [43][69], pour évaluer précisément les caractéristiques mécaniques (Fig. 59 et 60) et environnementales (Fig. 61) de ces nouveaux enrobés semi-tièdes produits à 90°C.

En parallèle à ces essais de laboratoire, trois expérimentations in-situ avec l'IFSTTAR de Nantes ont permis un inventaire du cycle de vie de ces réalisations à 90°C (cf. Fig. 62). [50][61][103]

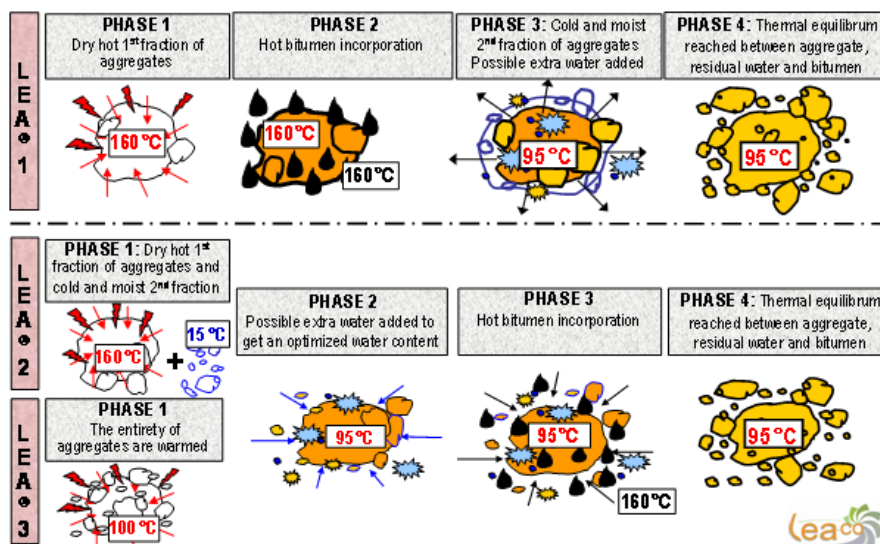


Figure 56. Procédés EBT® / LEA® couverts par les brevets européens EP1717369 et EP1712680. [115][116]

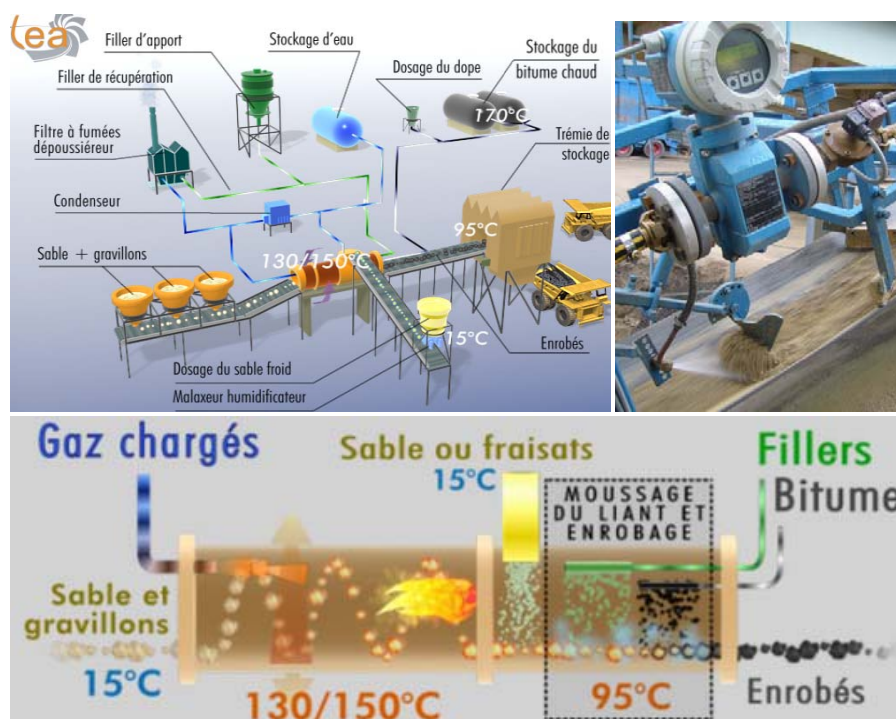


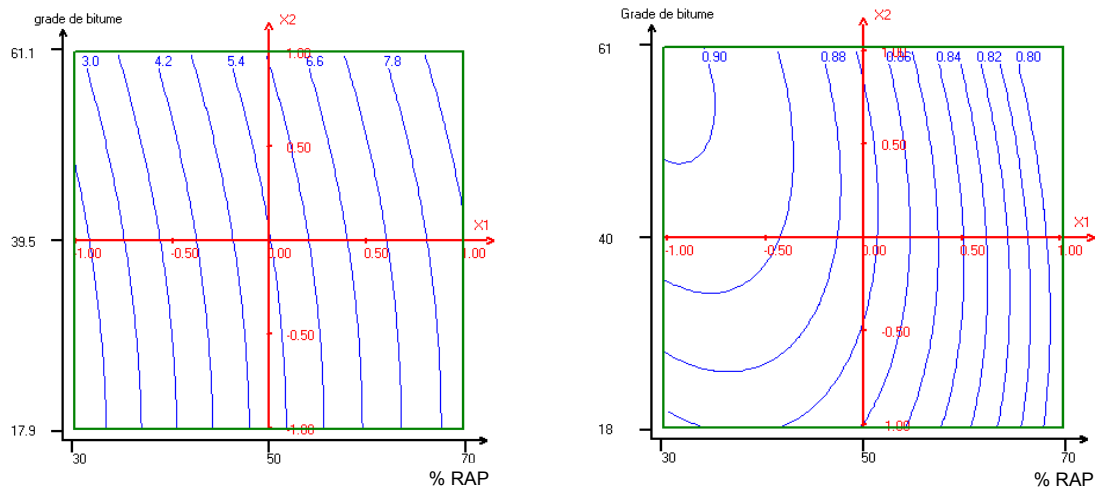
Figure 57. Process EBT® / LEA® (variante N°2) sur poste continu avec introduction à froid de sable ou fraisats humides dans l'anneau de recyclage, avec un contrôle d'humidité initiale des matériaux introduits à froid.



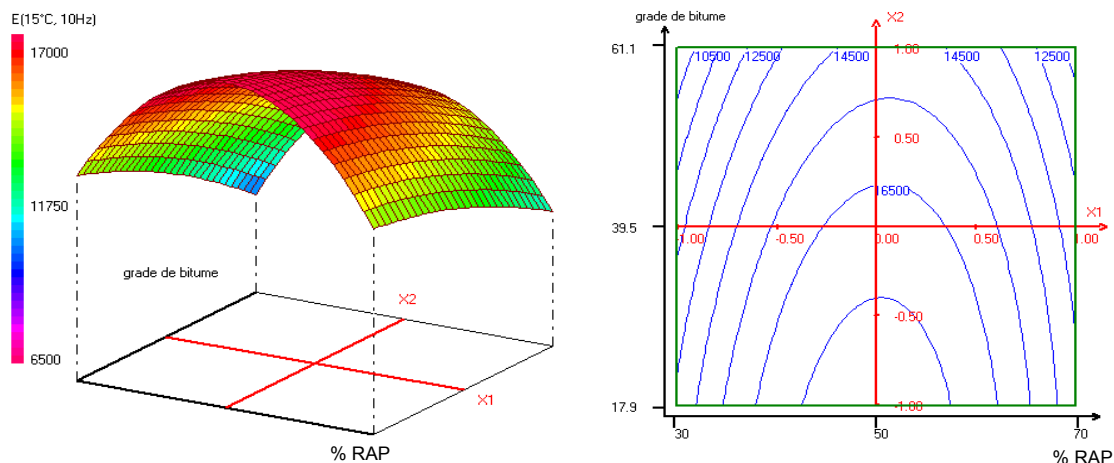
Figure 58. Exemple de chantier LEA® de 3100 tonnes (aérodrome vers Béziers, France).



### Caractéristiques mécaniques



**Figure 59.** Optimisation a) de l'aptitude au compactage (% vides PCG à 120 girations) et b) de la tenue à l'eau (ratio Duriez) selon le %RAP et le grade de bitume neuf, d'un EME2 0/14 calcaire LEA®. [46][47]



**Figure 60.** Module 15°C-10Hz d'un EME2 0/14 calcaire LEA® selon le %RAP et le grade de bitume neuf. [46][47]

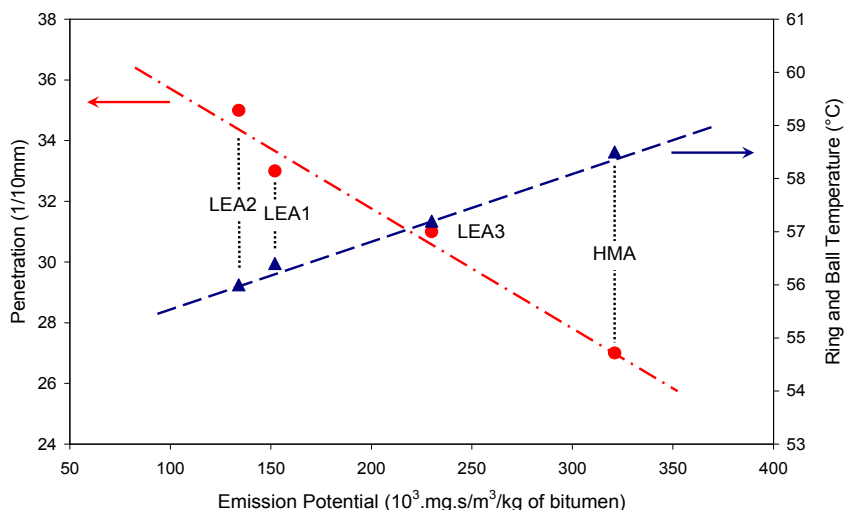
Les Figures 59 et 60 illustrent la nécessaire optimisation en laboratoire de la composition des enrobés semi-tièdes à fort taux d'agrégats d'enrobés recyclés (RAP). Contrairement à la Figure 45 obtenue dans le cadre de la formulation d'enrobés à chaud, en particulier, la Figure 60 illustre un moindre mélange des deux bitumes (bitume neuf et bitume issu du recyclé, ce dernier étant mal mobilisé lors de l'enrobage à température réduite). Malheureusement, l'essai de module complexe ayant été ici réalisé à 15°C-10Hz seulement, l'exploitation d'isothermes et d'isochrones comme en section 4.4.2 n'a pas été possible. Ce point reste à approfondir à l'avenir à l'aide du modèle 2S2P1D.

D'autres études en laboratoire, où le taux de RAP est compris entre 0% et 40%, mettent en exergue le nécessaire contrôle de l'humidité des matériaux introduits à froid lors du procédé semi-tiède à 90°C. Cette humidité initiale influence en effet la tenue à l'eau ultérieure de l'enrobé (ratio Duriez). [40]

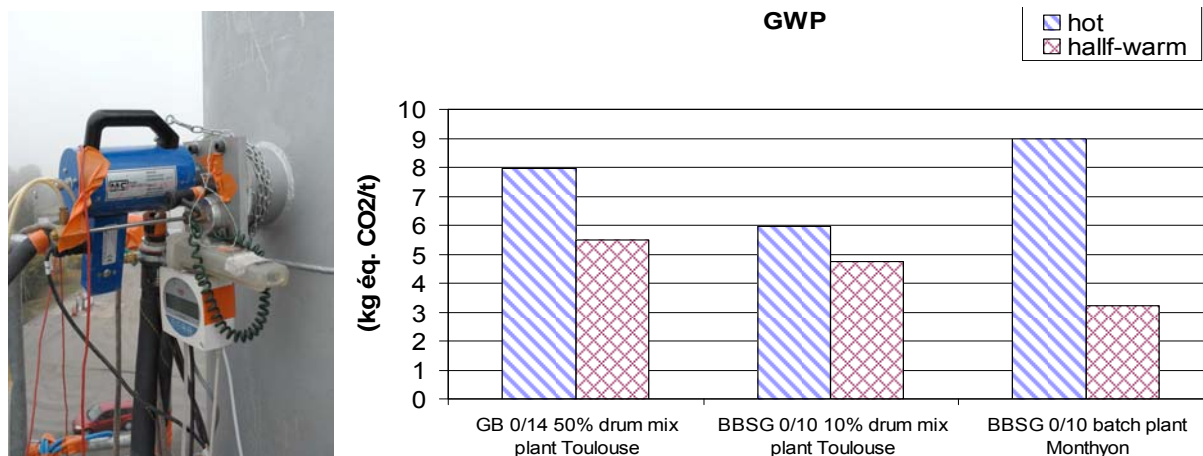
La durabilité d'enrobés semi-tièdes, de type grave-bitume GB3 (sans RAP), fut par ailleurs étudiée et jugée satisfaisante par la réalisation d'essais de fatigue à 10°C-25°C sur éprouvettes cylindres. [43]

### Caractéristiques environnementales

La Figure 61 illustre un résultat original, obtenu dans le cadre d'une collaboration avec Vincent Gaudfroy (IFSTTAR Nantes) : le potentiel d'émission de COG (composés organiques gazeux) lors d'une fabrication d'enrobé en laboratoire est corrélé linéairement avec l'évolution des valeurs de pénétrabilité et de température Bille-Anneau du bitume après enrobage. Ladite figure met donc en évidence une post-distillation du bitume selon le process et la température d'enrobage usité.



**Figure 61.** Pénétrabilité et température Bille-Anneau en fonction du potentiel d'émission de COG (HMA=chaud). [44][48][49][60]



**Figure 62.** (gauche) Echantillonnage des émissions au droit de la cheminée du poste d'enrobage ; (droite) Effet du process d'enrobage à chaud ou semi-tiède sur les gaz à effet de serre émis. [61]

La Figure 62 présente enfin une dernière contribution, réalisée alors en collaboration avec l'équipe d'Agnès Jullien et de Philippe Tamagny (IFSTTAR Nantes). Des expérimentations grandeur réelle sur postes d'enrobage ont permis d'effectuer un inventaire du cycle de vie et en particulier d'évaluer l'effet du process d'enrobage à chaud ou semi-tiède sur les gaz à effet de serre émis [61]. Une réduction de l'émission en eq.CO<sub>2</sub> allant de 20% à 55% est obtenue grâce au process EBT<sup>®</sup>/LEA<sup>®</sup>.

#### 4.6.3 Articles présentés en Annexe 6

Deux articles de rang A sont fournis en Annexe 6, pour illustration des travaux concernant les enrobés semi-tièdes:

- A. Romier, M. Audeon, J. David, Y. Martineau, F. Olard, Low-Energy Asphalt with Performance of Hot-Mix Asphalt, Transportation Research Record: Journal of the Transportation Research Board, No. 1962, Transportation Research Board of the National Academies, Washington D.C., pp. 101-112, 2006.
- V. Gaudefroy, F. Olard, B. Cazacliu, C. de La Roche, E. Beduneau, JP. Antoine, Laboratory investigations of mechanical performance of foamed bitumen mixes that use half-warm aggregates, Transportation Research Record: Journal of the Transportation Research Board, No. 1998, Transportation Research Board of the National Academies, Washington DC, pp. 89-95, 2007.

Les deux brevets européens EP1717369 et EP1712680 sont fournis en Annexe 6.



# Chapitre V. Perspectives de recherche et de diffusion scientifique

A travers ce mémoire et les six thématiques présentées en sections 4.1 à 4.6, j'ai voulu mettre en exergue les relations pouvant exister entre la composition, la structure et les propriétés thermo-mécaniques des principaux matériaux bitumineux utilisés en construction ou entretien routiers.

Pour conclure le mémoire, je présente ici quelques axes de recherche que je souhaite développer désormais, certains travaux ont déjà commencé et viendront en compléments des éléments introduits aux sections 4.1 à 4.6.

A ce jour, je prévois de continuer à développer en parallèle quatre grandes thématiques :

- le rôle des granulats et des fines,
- le rôle du liant bitumineux,
- le comportement in-situ des structures de chaussées,
- le rôle du process de fabrication vis-à-vis des performances et de l'impact environnemental.

Les deux premières thématiques ont trait à la formulation en laboratoire des matériaux bitumineux, tandis que la troisième thématique concerne le suivi de l'évolution des matériaux et des structures en place. La quatrième thématique, quant à elle, n'est plus seulement technique, mais fait également appel à des savoir-faire industriels et des notions environnementales.

## 5.1 Rôle des granulats et des fines

### **5.1.1 Diffusion de la technique GB5<sup>®</sup> à l'international**

L'avancée à la fois technique, économique et environnementale de la technique GB5<sup>®</sup> –trois décennies après l'invention et l'utilisation en France des EME2– arrive au moment précis où les Premium Asphalt Base Courses (durables et d'épaisseur réduite) deviennent incontournables dans de nombreux pays où le trafic Poids Lourds (PL) croît fortement. Cette diffusion à l'international des EME2 est l'occasion de confronter la technique GB5<sup>®</sup> à des diverses natures de granulats et de bitumes, à des climats et des trafics variés.

Par le biais de collaborations internationales avec le monde académique ou avec des entreprises routières, nous pourrions conforter nos acquis. La qualité des granulats, généralement moindre qu'en France (en termes d'angularité, de coefficient d'aplatissement, d'argilosité, de variabilité des caractéristiques de fabrication en carrière, etc.), sera à prendre en compte de façon toute particulière.

Aux Etats-Unis, les Universités du Minnesota et de Floride (les Prof. Mihai Marasteanu et Rey Roque, respectivement) sont intéressées par la technique d'optimisation granulaire GB5<sup>®</sup>. L'éventualité d'une année sabbatique de Mihai Marasteanu à l'ENTPE sur la modélisation 2S2P1D et la formulation GB5<sup>®</sup> a été évoquée en 2012, mais repoussée du fait de la crise économique –très forte– dans son Etat.

Des partenariats sont engagés dans l'hémisphère sud : l'éventualité d'une thèse co-encadrée par le Prof. Morris de Beer (Council for Scientific and Industrial Research (South Africa, Pretoria)) et le Prof. Hervé Di Benedetto, avec le soutien de l'Ambassade de France en Afrique du Sud est en discussion. Par ailleurs, les associations professionnelles SABITA (Afrique du Sud) et AAPA (Australie), ainsi que leurs membres, ont chacune manifesté leur intérêt. Des licences de brevet sont à cet effet prévues.

### 5.1.2 Continuation des recherches sur l'optimisation granulaire

Une collaboration avec l'équipe du Prof. Christophe Petit (Université de Limoges) a été entamée : le module complexe en cisaillement de plusieurs bitumes a été déterminé au moyen du rhéomètre à cisaillement cyclique DSR d'EIFFAGE Travaux Publics à Corbas sur un large spectre de fréquences et de températures. L'équipe de C. Petit effectue les essais sur enrobés à courbes granulaires typiques continues, d'une part, et à courbes granulaires discontinues optimisées (type GB5<sup>®</sup>), d'autre part, puis effectue l'analyse en éléments discrets pour vérifier ou non la prédiction du comportement visco-élastique linéaire de l'enrobé à partir de celui du liant.

En outre, avec l'équipe de Ferhat Hammoum (IFSTTAR Nantes), nous projetons de caractériser par analyse d'images 2D dans un 1<sup>er</sup> temps (triangulation de Delaunay, discrétisation du type Dirichlet-Voronoi) les empilements granulaires continus et discontinus optimisés: nombre de (pseudo-)contacts entre granulats, nombre de voisins, serrage de la classe dominante 10/14mm, coefficient de forme, orientation, répartition des vides. A partir de ces caractéristiques morphologiques, nous souhaitons établir une corrélation avec le comportement thermo-mécanique de l'enrobé. Un avenant scientifique (N°6) a ainsi été établi courant 2012 entre nos directions générales.

De plus, un consortium regroupant notamment EIFFAGE, les Universités de Lyon (H. Di Benedetto), de Limoges (C. Petit) et de Clermont-Ferrand (M. Grédiac et E. Toussaint), ainsi que l'IFSTTAR (F. Hammoum), prépare actuellement un dossier pour le prochain appel ANR MAT&PRO (matériaux et procédés, dépôt du dossier avril 2013), en se focalisant spécifiquement sur le rôle des interfaces entre granulats à l'échelle mésoscopique et entre couches de chaussées à l'échelle macroscopique. Ce projet complétera les travaux commencés avec C. Petit et F. Hammoum. H. Di Benedetto et son équipe prévoient deux apports : *i*) l'utilisation d'un micro-tomographe à l'Ecole Centrale de Lyon (ECL) pour mener une analyse 3D de l'empilement granulaire et *ii*) la mise au point d'un essai en torsion-compression sur cylindre creux pour tester l'interface entre deux couches d'enrobés (la texture de l'enrobé pouvant être un paramètre d'importance). M. Grédiac et E. Toussaint proposent une analyse par mesure des champs (déplacement et température, donnant accès respectivement au champ de déformations et à la dissipation visqueuse). L'exploitation de l'ensemble des résultats expérimentaux permettra d'établir des liens entre réponses locale et globale du matériau et de la structure de chaussée. La démarche de notre consortium est soutenue par le pôle de compétitivité VIAMECA.

Plus largement, grâce à l'ensemble de ces travaux sur l'optimisation de la composition granulaire des enrobés, je souhaite indirectement pouvoir optimiser les caractéristiques asymptotiques  $E_0$  (module statique) et  $E_\infty$  (module vitreux) dans le domaine linéaire (Equations 31 et 32, Figures 12 et 13). Augmenter le module statique  $E_0$  de l'enrobé est primordial puisque cela revient en pratique à redonner ses lettres de noblesse à l'enrobé dans le cas de charges lentes et lourdes (applications portuaires et aéroportuaires, etc).

Enfin, en dehors du domaine des enrobés bitumineux, j'envisage transposer de nouveau la méthode d'optimisation granulaire proposée aux formulations de bétons de ciment et aux MTLH (matériaux traités aux liants hydrauliques) utilisés en technique routière. Un stage de Master II est d'ores et déjà prévu au laboratoire central d'EIFFAGE Travaux Publics à Ciry-Salsogne (02) courant 2013.

### 5.1.3 Influence des fines

Jusqu'à présent, mes travaux de recherche n'ont pas intégré l'influence des fines (nature et granulométrie) sur les performances des matériaux bitumineux. La mesure et la modélisation du comportement visco-élastique linéaire des mastics (bitume+fines) pourraient s'avérer une étape intermédiaire cruciale dans le passage liant-enrobé (Equation 32). De plus, l'influence des fines sur le comportement en fatigue des enrobés est mal maîtrisée.

Une thèse confidentielle sur ce sujet, en collaboration avec un partenaire industriel et l'équipe du Prof. H. Di Benedetto, sera réalisée entre fin 2012 et mi-2016.

## 5.2 Rôle du liant bitumineux

### **5.2.1 Liants modifiés en polymère**

Depuis la fin des années 1980, la modification des bitumes par adjonction d'élastomères s'est généralisée dans les formules d'enrobés pour couches de roulement de chaussées. L'élasticité (réversibilité en particulier), la ductilité, la résistance à la fissuration et le moindre vieillissement du bitume ainsi modifié, sont devenus des avantages primordiaux dans le contexte actuel d'augmentation et d'agressivité accrue du trafic.

L'invention de la grave-bitume à hautes performances GB5<sup>®</sup>, rendant économiquement possible l'utilisation de polymère en couches d'assise et de liaison, ouvre un nouveau champ d'application des polymères. Rigidité et résistance en fatigue de l'enrobé ainsi formulé seront les deux paramètres à maximiser de façon prioritaire.

Un volet de ma recherche consistera, par ailleurs, à développer des liants modifiés à phase continue polymère. Dans le cas des copolymères de Styrène et Butadiène, au-delà de 5 à 6%, une inversion de phase s'opère permettant au polymère (gonflé de la phase malténique du bitume) de devenir la phase continue du liant au sein de laquelle se trouve une dispersion de bitume appauvri en maltènes (enrichi en asphaltènes). Une telle morphologie de liant modifié confère aux enrobés une très longue durée de vie (phases d'entretien espacées), i.e. des propriétés mécaniques ou de surface (macrotecture, mégatecture, uni) pour un enrobé de couche de roulement, pérennes dans le temps. Le point de départ de ce type d'innovation est la formulation spécifique du liant utilisé pour la couche de roulement du Viaduc de Millau, cette dernière subissant des niveaux de sollicitations, de déformations en particulier, jusqu'à cinq fois supérieurs aux niveaux rencontrés sur chaussées classiques, mêmes autoroutières. Depuis lors, l'optimisation de la formulation des liants modifiés à phase continue polymère (du point de vue technique et économique à la fois) doit redevenir l'une de mes priorités.

Enfin, dans un contexte haussier du prix des polymères d'origine pétrochimique et de sécurisation des approvisionnements, deux voies de recherche sont envisagées : l'évaluation de polymères alternatifs à bas coût ou recyclés, d'une part, ou de bio-polymères, d'autre part.

### **5.2.2 Degré de mobilisation du liant recyclé**

La collaboration scientifique avec BP et l'ENTPE, dans le cadre de la thèse de Salvatore Mangiafico, doit continuer jusqu'à fin mai 2014. D'ici là, un effort important sera mené pour apporter une analyse critique des essais de fatigue normalisés réalisés à ce jour. Une évaluation des effets biaisants (thixotropie, échauffement) sera déterminée pour des enrobés contenant un pourcentage de recyclés compris entre 0 et 60% et pour des bitumes neufs d'apport de natures différentes (rectifiés à l'air ou non en raffinerie, avec ou sans modification en polymères).

Un consortium regroupant l'USIRF (Union des syndicats de l'industrie routière française), l'ENTPE (H. Di Benedetto), l'IFSTTAR (P. Marsac), ainsi que des maîtrises d'ouvrages, prépare actuellement un dossier pour le prochain appel ANR VMD (villes et mobilités durables, dépôt du dossier février 2013), en se focalisant spécifiquement sur le degré de mobilisation du liant recyclé lors des fabrications à chaud ou tiède. La problématique particulière du multi-recyclage sera abordée en laboratoire et in-situ : est-il possible de recycler le bitume à plusieurs reprises? Simon Pouget (EIFFAGE Travaux Publics) pilotera au nom de l'USIRF ce consortium. Ma participation à quelques tâches (concernant l'analyse thermo-mécanique) est possible, en parallèle de mon travail avec BP et l'ENTPE. La démarche de notre consortium est soutenue par le pôle de compétitivité ADVANCITY.

## 5.3 Comportement in-situ des structures de chaussées

### **5.3.1 Evolution du comportement in-situ des chaussées autoroutières**

Le dimensionnement des entretiens de chaussées utilise la méthode française de dimensionnement des chaussées neuves. Cette méthode est utilisée en effet par défaut d'existence d'une méthode de dimensionnement des entretiens de chaussées. Pour optimiser la planification des travaux et le dimensionnement des solutions d'entretien, le groupe APRR (Autoroutes Paris Rhin Rhône), en partenariat avec l'Université de Lyon (H. Di Benedetto) et EIFFAGE Travaux Publics (S. Pouget et

moi-même) a initié une étude visant à déterminer l'état réel de son réseau et l'évolution de son endommagement, dans le cadre de la thèse CIFRE de Philippe Gaborit (2012-2015). Ce travail comprend un aspect expérimental qui se répartit entre l'expérimentation in-situ (curviamètre, ovalisation, prélèvements de blocs, instrumentation de sections tests) et l'expérimentation en laboratoire (module complexe, fatigue). L'analyse de ces résultats avec le trafic mesuré est en cours.

### **5.3.2 Instrumentation des chaussées in-situ**

Mon travail de recherche a jusqu'ici été focalisé essentiellement sur la caractérisation et l'optimisation des performances des matériaux bitumineux en laboratoire. L'évaluation du comportement in-situ par le biais d'instrumentation des matériaux et des interfaces est une des problématiques majeures que je souhaite traiter à court et à moyen terme.

Pour ce faire, dans le cadre de la thèse de P. Gaborit, nous avons entamé le suivi in-situ de deux sections autoroutières respectivement en EME2 et en GB5, aux comportements similaires en laboratoire malgré leurs formulations respectives aux antipodes l'une de l'autre. Deux technologies de capteurs de déformations (longitudinale, transversale et verticale) sont utilisées : des capteurs résistifs et des capteurs fibre optique.

Enfin, au sein du consortium soutenu par le pôle VIAMECA et regroupant EIFFAGE, les Universités de Lyon, de Limoges et de Clermont-Ferrand, et l'IFSTTAR, le manège de fatigue de l'IFSTTAR de Nantes et l'instrumentation de sections sur routes ou autoroutes seront autant d'occasions de développer nos savoir-faire et nos connaissances sur l'utilisation de capteurs de déformations et de température. Deux PME innovantes développeront avec nous, au sein de ce consortium, deux technologies sous couvert de confidentialité. Le caractère innovant de cette recherche collaborative réside encore dans la démarche de caractérisation et de modélisation multi-échelle des matériaux bitumineux (échelle micro ↔ méso ↔ macro ↔ échelle de la structure).

## **5.4 Rôle du process de fabrication**

L'influence du process de fabrication vis-à-vis des performances de l'enrobé bitumineux et de l'impact environnemental en centrale d'enrobage, est conséquent. Les techniques récentes d'enrobage tiède (100 – 140°C) ou semi-tiède (<100°C), ou encore de recyclage à très fort taux (jusqu'à 70%), illustrent l'intérêt commun des spécialistes de la Technique, du Matériel et des Industries, de se rapprocher pour mener des recherches collaboratives. Telle est ma conviction et l'engagement personnel que je me fixe.

Par ailleurs, l'évolution conjointe des techniques et des matériels d'enrobage de ces dernières années, pour des performances accrues et pour réduire l'empreinte environnementale, est à ce titre très encourageante.

## **5.5 Autres pistes**

La recherche se doit de fournir des éléments de robustesse indispensable à la maîtrise du progrès par une conscience pluridisciplinaire. Elle doit se faire de façon partenariale, en associant autant que faire se peut les milieux industriels et académiques, mais en associant aussi les spécialistes de la Chimie, de la Physique, de la Minéralogie, etc.

Osons innover également dans nos pratiques et nos procédures :

- les échanges de personnels entre organismes publics de recherche et entreprises ne sont que trop rarement pratiqués. Encourageons l'accueil de chercheurs d'organismes publics de recherche au sein des entreprises, par le biais d'années sabbatiques.
- Imaginons des structures communes (laboratoires mixtes public+privé) ; rappelons l'existence passée du centre NTEC (Nottingham Transportation Engineering Centre), commun à l'Université de Nottingham et à SHELL.

C'est du moins l'approche que je vais m'efforcer de poursuivre dans les années à venir.

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# **Mémoire d'Habilitation à Diriger des Recherches**

présenté par

François OLARD

## **Annexes**

**Relations entre la composition, la structure et les propriétés thermo-mécaniques des matériaux bitumineux**



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# Annexe 1

## Comportement thermo-mécanique des enrobés bitumineux à basses températures. Relation entre les propriétés du liant et de l'enrobé (2000-2003)

Deux articles de rang A sont fournis pour illustrer l'originalité du travail de thèse (2000-2003):

- F.Olard, H.Di Benedetto, General "2S2P1D" model and relation between the linear viscoelastic behaviors of bituminous binders and mixes, Road Materials and Pavement Design, Volume 4, Issue 2, 2003.
- F.Olard, H.Di Benedetto, The "DBN" Model: A Thermo-Visco-Elasto-Plastic Approach for Pavement Behavior Modeling. Application to Direct Tension Test and Thermal Stress Restrained Specimen Test, Journal of the Association of Asphalt Paving Technologists (AAPT), Vol.74, pp.791-828, 2005.



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## General “2S2P1D” model and relation between the linear viscoelastic behaviors of bituminous binders and mixes

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*ABSTRACT.* A research including a large experimental campaign on the characterization of the viscoelastic behavior of different bituminous materials was developed. The aim is to establish the links between the viscoelastic properties (which are observed in the small strain domain) of binders and those of bituminous mixes. The viscoelastic behavior of bituminous binders and mixes has been studied by performing complex modulus tests at different temperatures and frequencies. A unique rheological model has been developed for the modeling of linear viscoelastic properties of both bituminous binders and mixes. This model consists of a generalization of the Huet-Sayegh analogical model. Analyses on test data for nine different binders and four mixes, with one mix design, show that the introduced model fits quite well the measurements. Finally, from our first results, a transformation that is independent of the introduced model allows to predict easily and efficiently the mix complex modulus from the binder one.

*RÉSUMÉ.* Une recherche expérimentale sur le comportement viscoélastique de différents matériaux bitumineux a été menée. Le but est d'établir les liens existants entre les propriétés viscoélastiques (qui sont observées dans le domaine des petites déformations) des liants et celles des enrobés bitumineux. Le comportement viscoélastique des liants et des enrobés a été évalué à partir d'essais de module complexe à différentes températures et fréquences. Un modèle rhéologique unique a été développé pour la caractérisation des propriétés viscoélastiques linéaires des liants et des enrobés. Ce modèle est une généralisation du modèle analogique de Huet-Sayegh. D'après nos résultats sur neuf liants et quatre enrobés, pour une formulation d'enrobé fixée, le modèle introduit convient bien aux liants et aux enrobés bitumineux. Finalement, à partir de nos premiers résultats, une transformation indépendante du modèle permet de prédire facilement et efficacement le module complexe de l'enrobé à partir de celui du liant.

*KEYWORDS:* bitumens, bituminous mixes, linear viscoelastic properties, modeling.

*MOTS-CLÉS:* bitumes, enrobés bitumineux, propriétés viscoélastiques linéaires, modélisation.

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### 1. Introduction

Within the framework of a partnership between the DGCIB of the ENTPE and the companies Appia, Eurovia and TotalFinaElf, a research including a large experimental campaign on the characterization of the viscoelastic behavior of different bituminous materials was developed. The goal of this paper is confined to the modeling of the linear viscoelastic properties (in the small strain amplitude domain) of bituminous binders and mixes when considering a small number of loadings. Figure 1 sums up the overall view of this study.

Pertinent links between the linear viscoelastic properties of binders and those of bituminous mixes have been latterly established by the two authors. We refer the reader to a companion paper which is entitled "Linear viscoelastic properties of bituminous binders and mixes at low and mid temperatures" [OLA 03b].

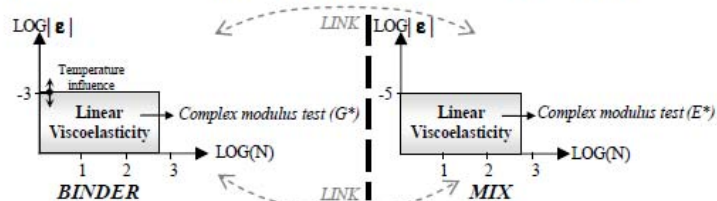


Figure 1. Linear ViscoElastic domains for binders and mixes in the axes  $\log(\text{strain})-\log(\text{number of applied cycles})$ .

As the complex modulus tests on bituminous mixtures are time-consuming, many researches have focused on the prediction of the mix complex modulus from the binder one so as to get approximate formulas that can be used in road design for the purpose of first estimates. Thus, a lot of work has already been presented in the literature to relate binder and mix moduli for different given mix designs ([SHE 91], [LYT 93], [FRA 95], [BOU 96], [DIB 96], [FRA 96], [BOU 97], [GOA 00], [ZEN 01], *etc.*). Yet, the relationships between binder and mix moduli found in the literature are only valid over a rather narrow range of frequencies and temperatures, or even only for binder and mix moduli measured in the same conditions of temperature and frequency. Some simple linear relationships between the norm of the complex shear modulus  $|G^*|$  of binder and the norm of the complex stiffness modulus  $|E^*|$  of mix have been found, for a given mix design. Notwithstanding, for very different conditions of temperature and frequency, these relationships are not valid any more.

Besides, most of the previous studies that deal with the rheological modeling of bituminous materials are focused on binders only [CHR 92] [MAR 99] [LES 96] [LES 02]. Both bituminous binders and mixes are seldom considered with the same rheological model [SAY 65] [ZEN 01].

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The paragraphs 2 and 3 respectively present the studied materials (nine bitumens and four mixes, with one mix design) and the experimental data of complex modulus tests at different temperatures and frequencies on both bituminous binders and mixes.

The paragraph 4 presents a non-exhaustive review of some of the rheological models that are used in the literature to model the Linear ViscoElastic behavior of bituminous binders and mixes. The experimental data on nine different binders and four mixes, for a given mix design, are then fitted with the different existing models. Some advantages and drawbacks of these models are pointed up.

The new general model (2S2P1D) which is valid for both bituminous binders and mixtures is presented in paragraph 5 and, finally, a new and simple relation is proposed in paragraph 6 to relate the complex moduli of binders and mixes.

## 2. Materials

Nine very different bitumens were selected to be investigated in order to study the influence of the binder : five hard pure bitumens (0/10, 10/20, 20/30, 35/50 and 50/70 penetration grade), two bitumens modified with a low content of elastomer, with or without crosslinking and two bitumens modified with a high content of elastomer or plastomer. The four polymer modified binders are named hereafter PMB1, PMB2, PMB3 and PMB4. It is also noteworthy that the PMB3 is a soft binder (cf. Table 1). Furthermore, it can be emphasized that the PMB3 and PMB4 correspond hereafter to highly modified binders in which the polymer is the continuous phase while the base bitumen is the discrete phase.

Table 1 presents the results of the conventional tests (Fraass brittle point, Penetration at 25°C and Softening Point Ring and Ball) initially performed on the different binders.

Four different bituminous mixes, made respectively from 10/20, 50/70, PMB3 and PMB4 bitumens with one type of aggregate and grading, have been tested. The mixture samples had a continuous 0/10mm diorite grading, a 3±1% void content and a binder content of 6% by weight of the aggregate.

binder	0/10	10/20	20/30	35/50	50/70	PMB1	PMB2	PMB3	PMB4
Level of modification						low	low	high	high
Pen 25°C (mm/10)	8	18	25	38	60	52	47	88	54
Softening Point R&B (°C)	88	62	60	53.5	48.5	57.5	55.4	100	65.8
Fraass (°C)	7	-10	-11	-15	-18	-16	-12	-20	-17

Table 1. Conventional results on the different binders.

### 3. Testing

#### 3.1. Complex modulus tests on binders

The complex modulus tests on binders were performed at the TotalFinaElf laboratory with a Metravib apparatus over a range of frequencies from 6,3 to 250Hz. From -30 to 30°C, compression/tension tests were conducted on cylindrical samples. From 30 to 60°C, the tests consisted of annular shearing of hollow cylindrical samples. The relation  $|E^*|=3|G^*|$  (incompressibility and isotropy) is supposed. It allows to plot all the results in function of  $G^*$  only.

The PMB's do not conform to the Time-Temperature Superposition Principle (TTSP) as their Black curves are not unique (Figure 2b). Meanwhile, a unique shear modulus master curves of  $|G^*|$  can be plotted at a reference temperature, hereafter named  $T_s$ , using a shifting procedure. Then, the value of  $|G^*|$  at any temperature can be obtained, provided the shift factor  $a_T$  is known. We call this property "Partial Time-Temperature Superposition Principle" (PTTSP) as the shifting procedure gives a unique and continuous master curve only for the norm of the modulus [OLA 03a] [OLA 03b]. Figure 3 illustrates this shifting procedure for the construction of the 50/70 master curve at  $T_s=10^\circ\text{C}$ . As regards the five pure bitumens, Figures 2a and 4 evidence a transition from a purely viscous terminal behavior (phase angle almost equal to  $90^\circ$ ) to a glassy plateau ( $|G^*| \approx 0.67\text{GPa}$ ). For PMB's (Figure 2b), the phase angle reaches a maximum value of around  $60^\circ$  and the norm of the complex modulus could reach an asymptotic minimum value (rubber-like elasticity at high temperatures). But this minimum value is very low (less than some KPa) and could be neglected in some modeling simulations.

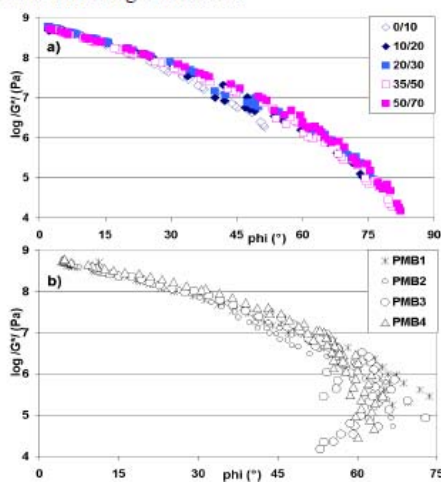


Figure 2. Black diagrams of the studied pure (a) and modified (b) bitumens.

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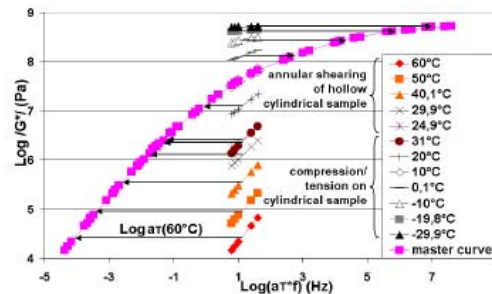


Figure 3. Construction of the 50/70 master curve at  $T_2=10^\circ\text{C}$ .

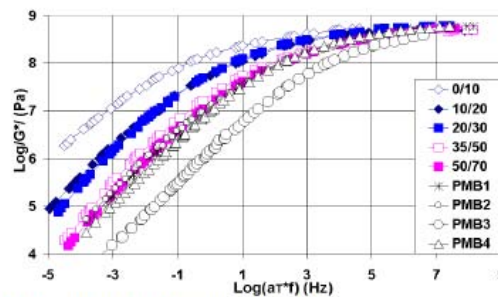


Figure 4.  $|G^*|$  master curves of binders at  $T_2=10^\circ\text{C}$ .

### 3.2. Complex modulus tests on bituminous mixes

A MTS hydraulic press was used at the Eurovia laboratory for a  $10^{-3}$  to 30Hz frequency range, at low temperatures (+15; 0; -15; -30°C). 220mm high cylindrical (diameter=80mm) specimens were cored from parallelepipedic slabs ( $L*W*H=60*40*15\text{cm}^3$ ) which were prepared using the LCPC slab compaction method (NFP98-250-2). The samples were tested in compression/tension. The sinusoidal cyclic loading was applied in stress mode. The axial strain measurement was obtained with three strain transducers placed at  $120^\circ$  around the sample. At each cycle, the strain was considered as the mean value of the measures given by the three transducers. This strain amplitude was checked to be lower than  $10^{-5}\text{m/m}$ , which is lower than the viscoelastic linearity limit of mixes at a level of around  $10^{-4}\text{m/m}$  as confirmed by [DOU 95] [DIB 98] [DIB 01] & [AIR 03].

The PMB3 and PMB4 mixes show thermorheologically complex behavior. The Time-Temperature Superposition Principle (TTSP) does not hold for the PMB3 and PMB4 mixes since their Black curves are not unique (Figure 5). Nevertheless, in a first approximation, stiffness modulus ( $|E^*|$ ) master curves can be plotted (Figure 6)

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by using a shifting procedure. As already observed for the modified binders, the "Partial Time-Temperature Superposition Principle" (PTTSP) also appears as quite well verified for the mixes in the considered time range.

The upper part of the  $|E^*|$  master curves approaches asymptotically to a unique maximum stiffness of the mixes which is around 43GPa for the considered mix design. In particular, that extremely high value may be due to the low binder content (3%) of these mixes. At higher temperatures and lower frequencies, the modulus approaches a limiting value which probably depends on the aggregate skeleton. As our experimental temperature range is confined to low temperatures (+15; 0; -15; -30°C), the minimum value of  $|E^*|$  can hardly be determined.

It may be underlined that the influence of bitumen aging during the production of mix samples has not been studied in this paper.

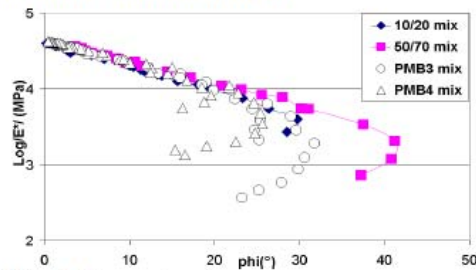


Figure 5.  $|E^*|$  Black diagram for mixes.

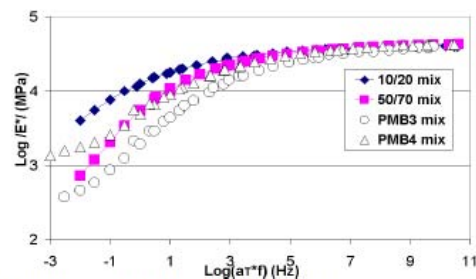


Figure 6.  $|E^*|$  master curves at  $T_r=15^\circ\text{C}$  for mixes.

#### 4. Some Linear ViscoElastic models proposed in the literature

##### 4.1. Discrete assembly of springs and dashpots – Discrete spectrum

Any combination of springs (elastic elements) and linear dashpots (Newtonian viscous elements) constitutes an analogical linear viscoelastic model [FER 80].

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Many particular combinations of such springs and linear dashpots have been used and presented in the literature, in view to describe in particular the linear viscoelastic behavior of bituminous materials (binders and mixtures). The simplest are the Maxwell and the Kelvin-Voigt models.

4.1.1. *The Maxwell and the Kelvin-Voigt models*

As regards the Maxwell and the Kelvin-Voigt models, a spring and a dashpot are respectively placed in series or in parallel. The representation of the Maxwell and the Kelvin-Voigt elements is shown in Figure 7.

The different equations of the creep function and the relaxation function of the two models are presented in Annex 2.

The two analogical models of Maxwell (viscoelastic liquid) and Kelvin-Voigt (viscoelastic solid) cannot describe correctly the complex behavior of bituminous materials [COST333 Report 99] but they may constitute the basic elements for more complex associations.

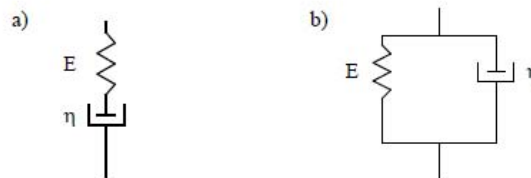


Figure 7. Maxwell (a) and Kelvin-Voigt (b) elements.

4.1.2. *The generalized Maxwell and Kelvin-Voigt models*

The generalized Maxwell model is constituted by a group of  $n$  Maxwell elements in parallel and possibly with a spring and a linear dashpot in parallel too. The generalized Kelvin-Voigt model is constituted by a group of  $n$  Kelvin-Voigt elements in series and possibly with one spring and one linear dashpot in series.

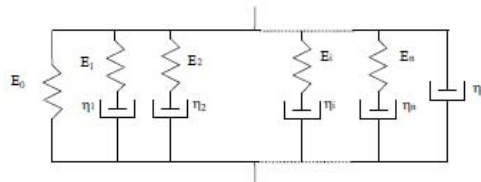


Figure 8. Generalized Maxwell model.

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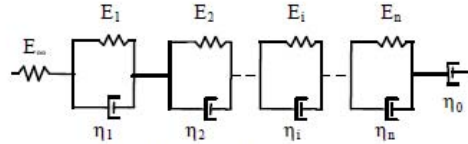


Figure 9. Generalized Kelvin-Voigt model.

The generalized Maxwell and Kelvin-Voigt models (Figures 8 and 9) allow a quite satisfactory description of the behavior of bituminous binders and mixes provided that a sufficient number  $n$  of elements is considered [LEE 96] [NEI 01] [LUN 02] ( $n=8$  appears as a minimum). The different equations of the creep function and the relaxation function of the two models are presented in Annex 2.

#### 4.2. Approaches considering a continuous spectrum

The generalized Maxwell or Kelvin-Voigt models with a discrete number of elements ( $n$ ) are not always appropriate for the description of a complex linear viscoelastic behavior. Nonetheless, when increasing  $n$ , the modeled behavior is enhanced. On the opposite, an infinite number of Maxwell or Kelvin-Voigt models characterized with a continuous spectrum is always appropriate (cf. Annex 2).

More generally, one should bear in mind that any linear viscoelastic material can be represented by any assembly eventually of an infinite number of Maxwell elements or Kelvin-Voigt elements respectively in parallel or in series [MAN 66].

The different models that have a continuous spectrum can be classified in two categories. First, the models in which the analytical expressions of either complex modulus or compliance creep function or relaxation function are used to fit experimental data. It is known that these three functions of frequency or time have mathematical relationships for Linear ViscoElastic materials [FER 80] [DIB 98] [MAR 99]. Second, "advanced" analogical models may be considered. Examples of these two kinds of models are given respectively in sections 4.2.1 and 4.2.2.

##### 4.2.1. Analytical expressions of Linear ViscoElastic functions

###### 4.2.1.1. Examples of analytical expressions for bituminous binders

The CA (Christensen-Anderson) model [CHR 92] and the CAM (Christensen-Anderson-Marasteanu) model [MAR 99] consist of mathematical formulations of the norm of the complex modulus  $G^*$  and the phase angle  $\delta$ , as follows :

$$|G^*(f, T)| = \frac{G_{\infty}}{[1 + (f_c/f)^k]^{n/k}} \quad [\text{Pa}] \quad [1a]$$

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$$\delta = \frac{90 m_e}{1 + (f_c / f)^k} \quad [^\circ] \quad [1b]$$

In equations [1a] and [1b],  $f$  is the frequency,  $f_c$  is a location parameter,  $G_g$  is the glassy shear modulus and  $k$  is a dimensionless shape parameter. The CA model corresponds to the particular case where  $m_e=1$ . In that case,  $f_c$ , also called crossover frequency, corresponds to the frequency at a given temperature where the storage and loss moduli are equal.

#### 4.2.1.2. Examples of analytical expressions for bituminous mixtures

In the framework of Superpave works, a version of the power law has been used throughout the report SHRP-A-357 [LYT 93] in modeling the linear viscoelastic behavior of bituminous mixes. The creep compliance or creep function, named  $J(t)$ , is as follows (and could be also used to model the behavior of binders in a first approximation):

$$J(t) = J_0 + J_1 t^m \quad [2]$$

where  $t$ : time

$J_0$ : creep compliance at zero time

$J_1, m$ : material constants

More recently, Zeng proposed the following generalization of the CAM model in equation [1a] which is valid both for bituminous binders and mixes [ZEN 01]:

$$|G^*(f, T)| = G_e + \frac{G_g - G_e}{[1 + (f_c / f)^k]^{m_e/k}} \quad [3]$$

In equation [3],  $G_e$  is the equilibrium shear modulus ( $f \rightarrow 0$ ),  $G_g$  is the glassy shear modulus ( $f \rightarrow \infty$ ),  $f_c$  is a location parameter,  $k$  and  $m_e$  are shape parameters, dimensionless.

The analytical expression giving the phase angle master curve is more complicated and not given here.

This model seems to be rather efficient but it presents for the modeling of binders ( $G_e=0$ ) the two same drawbacks that those of the CA and CAM models which are presented later in section 4.3.

#### 4.2.2. "Advanced" analogical models

The "advanced" analogical models with continuous spectrum (*i.e.* can be represented by an infinite number of Kelvin-Voigt elements in series or Maxwell elements in parallel) have a physical meaning and representation so that they are hereunder considered as more pertinent than the models in which empirical mathematical formulas are used to fit experimental data.



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#### 4.2.2.1. Parabolic element

A parabolic element is an analogical model with a parabolic creep function and consequently a stiffness complex modulus  $E^*$  as follows :

$$J(t) = a \left( \frac{t}{\tau} \right)^h \quad [4]$$

$$E^*(i\omega\tau) = \frac{(i\omega\tau)^h}{a\Gamma(h+1)} \quad [5]$$

with  $i$  : complex number defined by  $i^2 = -1$ ,

$\omega = 2\pi$ \*frequency, the pulsation,

$h$  : exponent such as  $0 < h < 1$ ,

$a$  : dimensionless constant,

$\Gamma$  : gamma function<sup>1</sup>,

$\tau$  : characteristic time, which value varies only with temperature.

$\tau$  is a function of temperature and accounts for the Time-Temperature Superposition Principle.  $\tau$  evolution may be approximated by a WLF type law [FER 80] in the range of temperature observed in laboratory [NEI 01] [OLA 03b] :

$$\tau(\theta) = a_T(\theta) \cdot \tau_0 \quad [6]$$

where  $a_T(\theta)$  : the shift factor at temperature  $\theta$ ,

$\tau_0 = \tau(T_s)$  determined at the reference temperature  $T_s$ .

The shift factor at temperature  $\theta$ , named  $a_T(\theta)$ , can be determined by means of equation [7]. For a given reference temperature  $T_s$ , the WLF equation [7] gives  $\log(a_T)$  as a function of three constants  $C_1$ ,  $C_2$ ,  $T_s$  and  $T$ [°C] :

$$\log(a_T) = \frac{-C_1(T - T_s)}{C_2 + (T - T_s)} \quad [7]$$

The values to be assigned to  $C_1$  and  $C_2$  are presented in Annex 1 for each studied bituminous material.

This model has a continuous spectrum since its representation by a generalized Maxwell (resp. Kelvin-Voigt) model needs a infinite number of Maxwell (resp. Kelvin-Voigt) elements.



Figure 10. Representation of the parabolic element.

<sup>1</sup>  $\Gamma(n) = \int_0^{\infty} t^{n-1} e^{-t} dt$  with  $n > 0$  or  $\text{Re}(n) > 0$

## 4.2.2.2. Huet model (proposed for binders)

The Huet model [HUE 63] was initially proposed for binders and mixes. It is an analogical model constituted by a combination of a spring (stiffness  $E_\infty$ ) and two parabolic elements ( $J_1(t)=at^h$  &  $J_2(t)=bt^k$ ) in series (Figure 11). This model has also a continuous spectrum, *i.e.* can be represented by an infinity of Kelvin-Voigt elements in series or Maxwell elements in parallel. The analytical expressions of the complex modulus and the creep function are given in Annex 2. No analytical expression of the relaxation function  $R(t)$  is available.

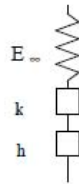


Figure 11. Representation of the Huet model [1963].

## 4.2.2.3. Huet-Sayegh model (proposed for bituminous mixes)

The Huet model does not fit very well the bituminous mixes response at very low frequencies and/or high temperatures. Indeed, from equation [31] in Annex 2, the complex modulus  $E^*$  tends towards zero when  $\omega\tau$  tends towards zero (at low frequencies and high temperatures), whereas the experimental modulus of bituminous mixtures approaches a limiting value  $E_0$  called static modulus that probably depends on the aggregate skeleton. Thus, the Huet model has been adapted by Sayegh [SAY 65] in adding a spring of small rigidity compared with  $E_\infty$  in parallel (Figure 12).

The complex modulus of the Huet-Sayegh model is given by the following expression :

$$E^*(i\omega\tau) = E_0 + \frac{E_\infty - E_0}{1 + \delta(i\omega\tau)^{-k} + (i\omega\tau)^{-h}} \quad [8]$$

with the same previous notations and  $E_0$  the static modulus when  $\omega\tau \rightarrow 0$ .

It is considered that the Huet-Sayegh model [SAY 65], which has a continuous spectrum, translates very correctly the linear behavior of bituminous mixes in the small strains domain for any range of frequencies and temperatures. This point was shown in different previous studies [SAY 65], [DEL 96], [NEI 97], [COST333 Report 99], [OLA 00] & [NEI 01].

Moreover, only six constants ( $\delta$ ,  $k$ ,  $h$ ,  $E_\infty^{\text{Say}}$ ,  $E_0^{\text{Say}}$  and  $\tau_0$ ) (equation [8]) are needed to entirely determine the linear viscoelastic behavior of the considered material at a given temperature. These six constants can be obtained by an

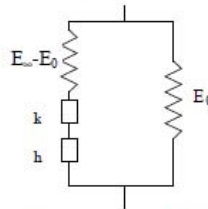
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optimisation process from the complex modulus tests results. Let's recall that the experimental results show that the behavior can be considered as linear viscoelastic only in the small strain domain.

A previous paper from Neifar et al. [NEI 01] dealing with a thermo-visco-plastic law developed at the ENTPE for bituminous mixtures (the Di Benedetto-Neifar "DBN" law) uses this model for the calibration of that thermo-visco-plastic law in the Linear ViscoElastic (LVE) domain.

If the static modulus  $E_0$  is equal to zero, the Huet and Huet-Sayegh models are identical.

Unlike the Huet model, no analytical expression of the creep function of the Huet-Sayegh model is available in the time domain.



**Figure 12.** Representation of Huet-Sayegh model [SAY 65],  $h$  and  $k$  are two parabolic creep elements.

#### 4.3. Drawbacks of the so far used continuous models

First, the models in which empirical mathematical formulas are used to fit experimental data can be considered as less pertinent than analogical models. In fact, the interest of the latter is to have a physical meaning and representation.

Second, let us consider the previous models introduced in section 4.2 at very low frequencies : the SHRP model (equation [2]), the CA and CAM models, the Zeng model, the parabolic element, the Huet model and the Huet-Sayegh model.

From equation [2], it can be underlined that the SHRP model consists of a spring and a parabolic element in series.

The CAM model (resp. CA model) is equivalent to a parabolic element (resp. a linear dashpot) when  $\omega \rightarrow 0$  (in the frequency domain) or  $t \rightarrow \infty$  (in the time domain). The same remark is also valid for the Zeng model, since it equals the CAM model for binders. Indeed, the CAM formulas [1a] & [1b] become :

$$G^*(if) \underset{f \rightarrow 0}{\approx} G_{\xi} f_c^{-m_s} (if)^{m_s} \quad [9a]$$

$$J^*(if) \underset{f \rightarrow 0}{\approx} G_{\xi}^{-1} f_c^{m_s} (if)^{-m_s} \quad [9b]$$

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Concerning the Huet model, it is equivalent to a parabolic element when  $\omega \rightarrow 0$ . This means also when  $t \rightarrow \infty$  for creep or relaxation. It is also noteworthy that, for pure binders ( $E_0=0$ ), the Huet-Sayegh model is equivalent to the Huet one.

Finally, at very low frequencies, it can be emphasized that all the presented models –except the CA model which is equivalent to a linear dashpot (power=1)– are equivalent to a parabolic element (eventually with a spring in series).

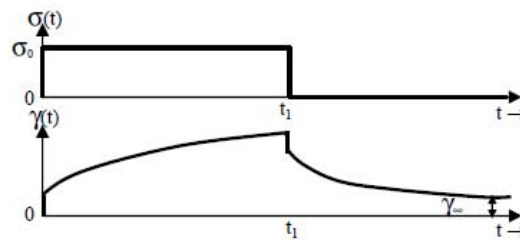
Then, for the previous models, the creep compliance function termed hereunder  $J(t)$  is as follows (only when  $t \rightarrow \infty$ ) :

$$J(t) \approx at^m \quad [10]$$

where  $t$  is time;  $a$  and  $m$  are constants.

## 4.3.1. Drawback in the modeling of the “fluid” characteristic of bitumens

A very noticeable drawback of the previous models –except the CA model– is that they cannot correctly model the fluid characteristic of bitumens. To show that result, let us consider the loading and response illustrated in Figure 13.



**Figure 13.** Example of shear creep and creep recovery in the case of the previous models ( $D_0=0$  except for the SHRP model), from equation [10].  $\gamma_\infty$  value is introduced in the text.

During a creep experiment, the stress is suddenly removed at a time  $t=t_1$ . The rate of strain will change sign. The course of this reverse deformation is called creep recovery. When considering Figure 13 and equation [10], after a sufficiently long time, the shear strain approaches a final value  $\gamma_\infty = \sigma_0 a m t_1^{m-1}$ .

If  $m < 1$ , the material is a parabolic element considered as viscoelastic solid since the recovery is complete ( $\gamma_\infty = 0$ ). The material returns toward its reference initial state (definition proposed by Ferry [FER 80]).

If  $m = 1$ , the material can be modeled by a linear dashpot and its behavior is liquid like. Indeed, the material has no favored reference state.

The case  $m > 1$  is not physically acceptable for bitumens because  $\gamma_\infty \rightarrow \infty$ . This would mean that the strain still increases up to infinity while no stress is applied.

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Therefore, amidst the previous models, only the CA model ( $m=1$ ) can be used to model viscoelastic fluids such as bitumens. An attempt of using the CA formulas [1a] and [1b] ( $m_s=1$ ) to model the linear viscoelastic behavior of the 50/70 pen binder is presented in Annex 3. Its drawbacks are underlined. Some of these drawbacks are also mentioned by Marasteanu [MAR 99].

4.3.2. Fitting of data on binders and mixes with the Huet-Sayegh model

An attempt of using the Huet-Sayegh model to fit the experimental data of the 50/70 binder and the corresponding mix is presented Figures 14 and 15.

As already underlined in section 4.2.2.3, the Huet-Sayegh model translates correctly the linear behavior of bituminous mixes in the small strains domain for any range of frequencies and temperatures. This point that was shown in different previous studies ([SAY 65], [DEL 96], [NEI 97], [COST333 Report 99], [OLA 00] & [NEI 01]) is highlighted in Figure 15 for the 50/70 mix. Notwithstanding, Figure 14 shows that this model (similar to the Huet model in that case since  $E_0=0$ ) is not suitable for binders at very low frequencies where it is equivalent to a parabolic element instead of a linear dashpot. As shown in Figures 14 and 15, the  $h$  (resp.  $k$ ) parameter of the Huet-Sayegh model is related to the ratio  $\Delta E_2/\Delta E_1$  (slope in the complex plane) when  $\omega\tau$  tends to 0 (resp. to infinity).

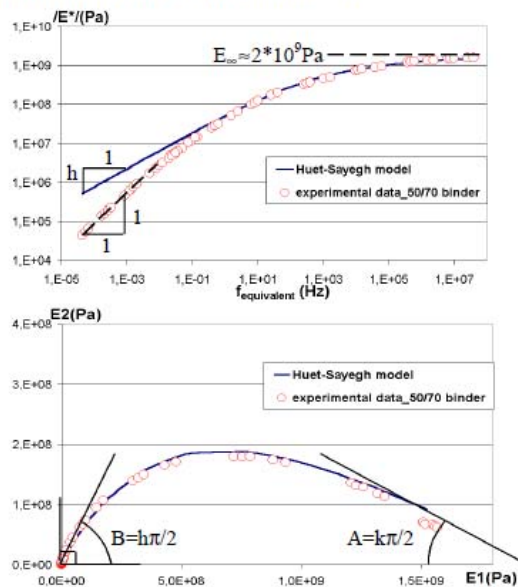
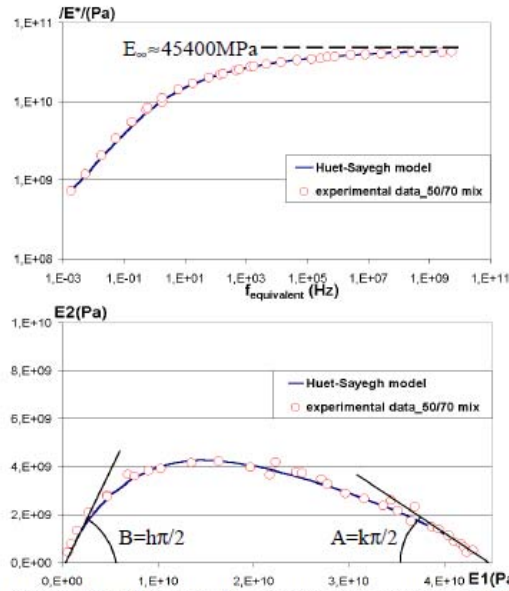


Figure 14. Comparison between the experimental complex modulus data on the 50/70 binder and the corresponding calibrated Huet-Sayegh model ( $T_z=10^\circ\text{C}$ ).

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**Figure 15.** Comparison between the experimental complex modulus data on the 50/70 mix and the corresponding calibrated Huet-Sayegh model ( $T_2=10^\circ\text{C}$ ).

Table 2 presents the calibrated parameters of the Huet-Sayegh model for the 50/70 bitumen and the corresponding mix. It has to be underlined that, as shown in section 5.2,  $\delta$ ,  $k$  and  $h$  have respectively the same values for binders and mixes.

parameters	$\delta$	$k$	$h$	$E_0$ (MPa)	$E_\infty$ (MPa)	$\text{LOG}\tau_0(T_2=10^\circ\text{C})$
50/70 binder	2.0	0.175	0.55	0	2000	-3.721
50/70 mix	2.0	0.175	0.55	250	45400	-1.414

**Table 2.** Parameters of the Huet-Sayegh model for the 50/70 bitumen and mix.

## 5. The proposed general model for bituminous materials

### 5.1. Formalism of the introduced general model "2S2P1D"

To avoid the drawbacks presented in the previous paragraphs, the authors present hereunder an alternative general model (generalization of the Huet-Sayegh model) valid for both the bituminous binders and mixtures.

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This general model is based on a simple combination of physical elements (spring, dashpot and parabolic element). The Huet-Sayegh model [SAY 65] has been adapted by adding a linear dashpot in series with the two parabolic elements and the spring of rigidity  $E_{\infty}-E_0$  (Figure 16). Thus, in particular for binders, for which  $E_0=0$ , the model is equivalent to a linear dashpot at very low frequency.

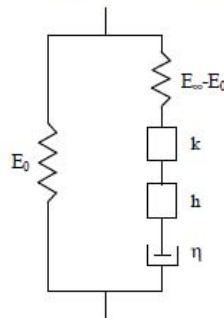
Like the Huet-Sayegh model, the introduced model has a continuous spectrum (*i.e.* can be represented by an infinity of Kelvin-Voigt elements in series or Maxwell elements in parallel).

At a given temperature, the introduced model has 7 constants and its complex modulus is given by the following expression :

$$E^*(i\omega\tau) = E_0 + \frac{E_{\infty} - E_0}{1 + \delta(i\omega\tau)^k + (i\omega\tau)^h + (i\omega\beta\tau)^{-1}} \quad [11]$$

with the same previous notations ( $0 < k < h < 1$ ) and also with a Newtonian viscosity  $\eta = (E_{\infty} - E_0)\beta\tau$ . When  $\omega\tau \rightarrow 0$ , then  $E^*(i\omega\tau) \rightarrow E_0 + i\omega(E_{\infty} - E_0)\beta\tau$ .  $\beta$  is dimensionless.

It has to be emphasized that only 7 constants ( $\delta$ ,  $k$ ,  $h$ ,  $E_{\infty}$ ,  $E_0$ ,  $\beta$  and  $\tau_0$  (equation [6])) are needed to entirely determine the linear viscoelastic behavior of the considered material, at a given temperature. For binders, the experimental static modulus is very close to zero. Thus, in equation [11],  $E_0$  can be taken equal to zero for binders and the number of constants of the model can be reduced to six.



**Figure 16.** Representation of the introduced general model "2S2P1D" for both bituminous binders and mixes,  $h$  and  $k$  are two parabolic creep elements.

Furthermore, if the hypothesis of a linear viscoelastic thermorheologically simple behavior can be applied to the considered material (*i.e.* the Time-Temperature Superposition Principle holds (TTSP)), only the  $\tau$  parameter depends on temperature.  $\tau$  evolution may be approximated by a WLF type law [FER 80] in the range of temperature observed in laboratory [NEI 01] [OLA 03b] (cf. equations [6] and [7]).

If the TTSP holds, the two additional constants  $C_1$  and  $C_2$  (calculated at the reference temperature  $T_s$ ) of the WLF equation [7] are needed. The number of constants of the model amounts to 9 (and 8 for binders since  $E_0=0$ ).

From our results on nine very different binders and four mixes (with one mix design), this comprehensive model translates correctly the linear viscoelastic behavior of bituminous binders and mixes in the small strains domain for any range of frequencies and temperatures. This point is shown in the next paragraph.

Unlike the Huet model and like the Huet-Sayegh model, no analytical expression of the creep function of this model is available in the time domain. Some simulations are shown in the next paragraph.

### 5.2. Calibration of the general model "2S2P1D"

The selected method to determine the constants of the "2S2P1D" model (equation [11]) consists in minimizing the sum of the distances between the experimental complex modulus and that of the introduced general model at  $N$  points of pulsation  $\omega_i$ . This minimization is made at a reference temperature  $T_s$  using the Solver feature of MS Excel. It comes :

$$\text{minimization of } \sum_{i=1}^{i=N} \left( \left[ E_1^{\text{exp}}(\omega_i) - E_1^{\text{model}}(\omega_i) \right]^2 + \left[ E_2^{\text{exp}}(\omega_i) - E_2^{\text{model}}(\omega_i) \right]^2 \right) \quad [12]$$

$E_1^{\text{exp}}$  and  $E_2^{\text{exp}}$  are respectively the real part and the imaginary part of the experimental complex modulus  $E^{*\text{exp}}$ .  $E_1^{\text{model}}$  and  $E_2^{\text{model}}$  are respectively the real part and the imaginary part of the complex modulus  $E^{*\text{model}}$  given by the "2S2P1D" model.

Figure 17 presents the comparison between the experimental complex modulus data on the 50/70 binder and the corresponding calibrated general model for the reference temperature  $T_s=10^\circ\text{C}$ . As regards the 50/70 mix, Figure 18 presents the comparison between the experimental complex modulus data and the corresponding calibrated model at the same reference temperature  $T_s=10^\circ\text{C}$ . These two figures evidence that the introduced "2S2P1D" model translates correctly the linear viscoelastic behavior of both bituminous binders and mixes in the small strains domain for any range of frequencies and temperatures. It is noteworthy that the model also works for PMB's and PMB mixtures (cf. Annex 4).

Nonetheless, Figures 17 and 18 show that the introduced general model does not fit perfectly the experimental data of phase angle when the latter is between  $50^\circ$  and  $70^\circ$  for the 50/70 bitumen, and between  $35^\circ$  and  $40^\circ$  for the 50/70 mix. As regards the 50/70 bitumen, this range of phase angle corresponds to the transition to the terminal linear dashpot when  $\omega\tau \rightarrow 0$ . Moreover, it must be emphasized that the fitting of the experimental data of binders and mixes could be enhanced by adding a third parabolic element in the model. This alternative has been given up to prevent the model from getting much more complex. Yet, the introduced model enables to predict correctly enough the linear viscoelastic behavior of this 50/70 bitumen and



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the 50/70 mix in the domain of low frequencies and high temperatures as well as in the domain of high frequencies and low temperatures, which is a good improvement of the so far used models.

In addition, it must be underlined that the difference between the experimental phase angle and the phase angle predicted by the model remains inferior to  $5.5^\circ$  (resp.  $3.8^\circ$ ) for the 50/70 pen bitumen (resp. 50/70 mix).

From our results on nine binders and four mixes, it can be concluded that the introduced model translates correctly the behavior of both bituminous binders and mixes. Due to the lack of place, the different curves are not provided herein, but the "2S2P1D" model fits the experimental data very well for both bituminous binders and mixes, whatever the considered frequency or temperature ranges.

Table 3 presents the different parameters of the introduced general model for the 10/20, 50/70, PMB3 and PMB4 binders and the four corresponding mixes (at  $T_s=10^\circ\text{C}$  for  $\tau_0$ ).

One should bear in mind that the PMB's only conform as a first approximation to the introduced "Partial Time-Temperature Superposition Principle" (PTTSP) at high temperatures. From our results, the PMB's conform to the "Time-Temperature Superposition Principle" (TTSP) only at temperatures below  $10^\circ\text{C}$ .

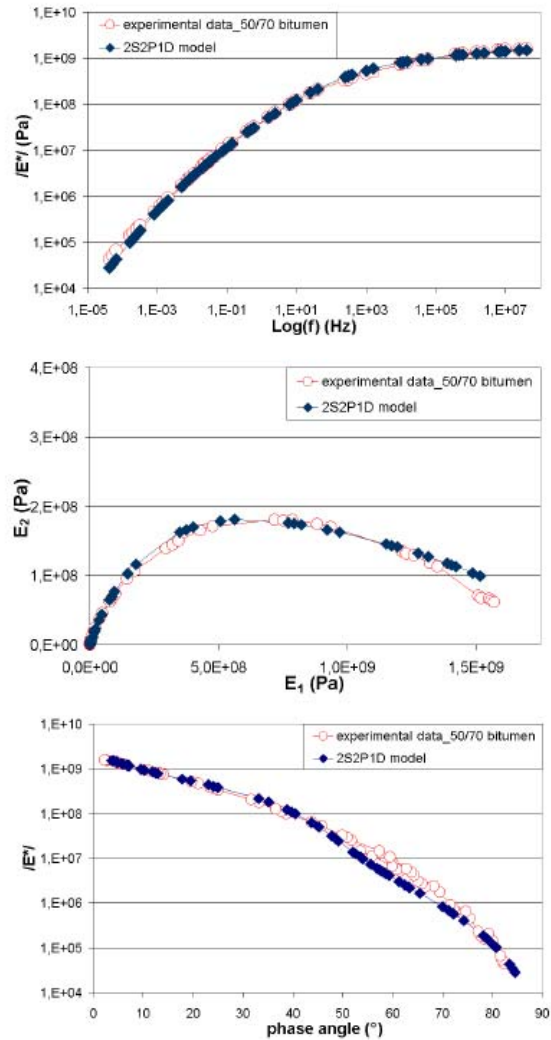
Table 3 points up that the four binders have close parameters (except for  $\tau$  at  $10^\circ\text{C}$  ( $\tau_0$ )). This result has also been checked with the five other bitumens. This point is also true for the four mixes. Indeed, as illustrated in Figure 19, it is confirmed when observing that the four mixtures have a quasi-identical representation in the Cole-Cole diagram.

Material	$\delta$	k	h	$E_0$ (MPa)	$E_\infty$ (MPa)	$\beta$	$\text{LOG}\tau_0(10^\circ\text{C})$
10/20 binder	2.2	0.19	0.52	0	2000	800	-2.608
50/70 binder	2.0	0.175	0.55	0	2000	320	-3.721
PMB3 binder	2.1	0.18	0.54	$0.01^2$	2000	1000	-5.260
PMB4 binder	1.7	0.18	0.46	$0.004^2$	2000	1500	-4.214
10/20 mix	2.2	0.19	0.52	490	41500	800	0.220
50/70 mix	2.0	0.175	0.55	250	45400	320	-1.414
PMB3 mix	2.1	0.18	0.54	250	44000	1000	-2.347
PMB4 mix	1.7	0.18	0.46	1050	43500	1500	-1.634

**Table 3.** Parameters of the introduced model for the 10/20, 50/70, PMB3 and PMB4 binders, and the four corresponding mixes.

<sup>2</sup>  $E_0 > 0$  in general for PMB's (rubber-like elasticity at high temperatures, cf. Figure 2). But these values are very small and can be neglected if need be.

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**Figure 17.** Comparison between the experimental complex modulus data on the 50/70 bitumen and the corresponding calibrated "2S2P1D" model ( $T_r=10^\circ\text{C}$ ). Constants are given in Table 3.

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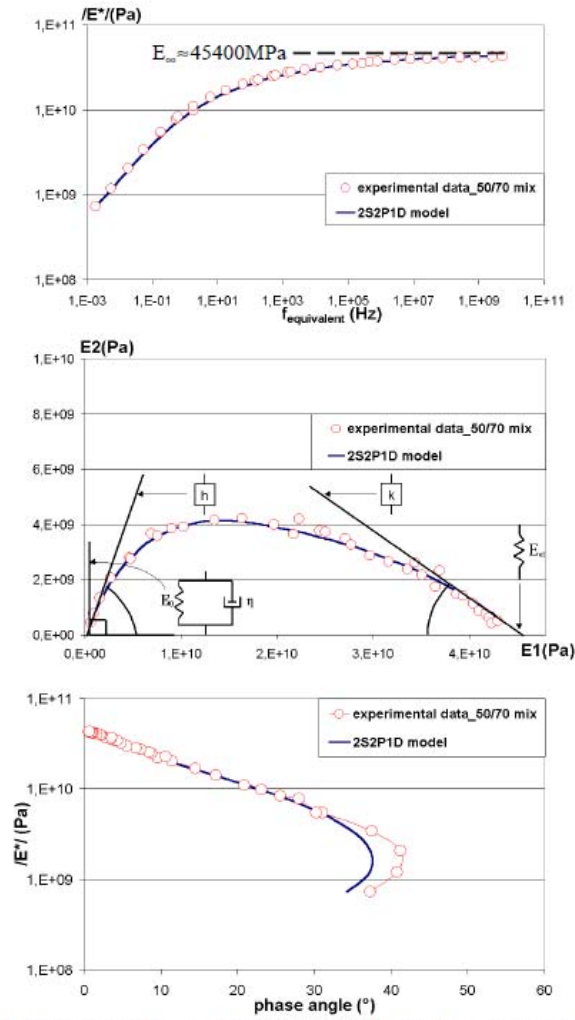


Figure 18. Comparison between the experimental complex modulus data on the 50/70 mixture and the corresponding calibrated "2S2P1D" model ( $T_i=10^{\circ}\text{C}$ ). Constants are given in Table 3.

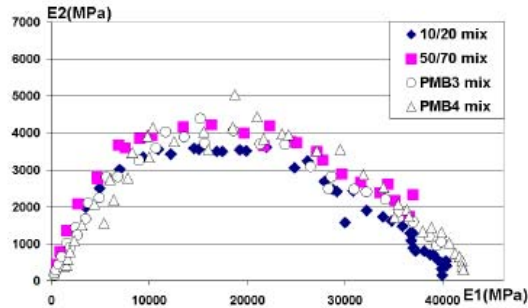


Figure 19. Complex moduli experimental data of the four mixes (made from 10/20, 50/70, PMB3 and PMB4 binders) represented in the Cole-Cole diagram.

Table 3 highlights that each mixture has the same parameters  $\delta$ ,  $k$ ,  $h$  and  $\beta$  (equation [11]) as the corresponding binder. The only different parameters between the binder and mix complex moduli are the static and glassy modulus ( $E_0$  and  $E_\infty$ ) and  $\tau_0$ . This confirms that the mix rheology directly depends on the binder rheology for the considered aggregate skeleton and binder content. Then, equation [11] gives the two following expressions respectively for the binder and mix complex moduli at low and mid temperatures :

$$E_{\text{binder}}^*(i\omega\tau_{\text{binder}}) = E_{0_{\text{binder}}} + \frac{E_{\infty_{\text{binder}}} - E_{0_{\text{binder}}}}{1 + \delta(i\omega\tau_{\text{binder}})^{-k} + (i\omega\tau_{\text{binder}})^{-h} + (i\omega\beta\tau_{\text{binder}})^{-1}} \quad [13]$$

$$E_{\text{mix}}^*(i\omega\tau_{\text{mix}}) = E_{0_{\text{mix}}} + \frac{E_{\infty_{\text{mix}}} - E_{0_{\text{mix}}}}{1 + \delta(i\omega\tau_{\text{mix}})^{-k} + (i\omega\tau_{\text{mix}})^{-h} + (i\omega\beta\tau_{\text{mix}})^{-1}} \quad [14]$$

where only  $\tau_{\text{binder}}$  and  $\tau_{\text{mix}}$  are functions of the temperature.

Besides, it must be emphasized that the glassy modulus  $E_{\infty_{\text{binder}}}$  of all binders can be considered as equal to  $2 \cdot 10^9$  Pa. It is very close to the values of glassy moduli found by Franken et al. [FRA 97] for eight binders ( $E_{\infty} \approx 2.157 \pm 0.198$  GPa). In addition, the static modulus  $E_0$  of binders can be supposed to be equal to zero.

As regards the four tested bituminous mixtures, their static (resp. glassy) modulus ranges from 250 to 1050 MPa (resp. from 41500 to 45400 MPa). The variation of the static modulus from one mix to another could be explained by the slight heterogeneity among the tested samples.

Indeed, the different mixture samples may have slightly different aggregate grading and/or void content ( $3 \pm 1\%$ ). As the static modulus (which is considered to represent the ultimate interlock between aggregates when the contribution of the binder in a mixture is negligible) probably depends on the aggregate skeleton and void content, this minimum asymptotic value may slightly vary.

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As illustrated in Figure 20, the logarithms of  $\tau_{0\text{binder}}$  and  $\tau_{0\text{mix}}$ , determined at  $T_c=10^\circ\text{C}$  respectively for bitumens and mixes, are well correlated ( $r^2=0.96$ ) :

$$\tau_{0\text{mix}} = 10^\alpha * \tau_{0\text{binder}} \quad [15]$$

with  $\alpha = 2.66$  which probably depends on mix design and ageing during mixing .

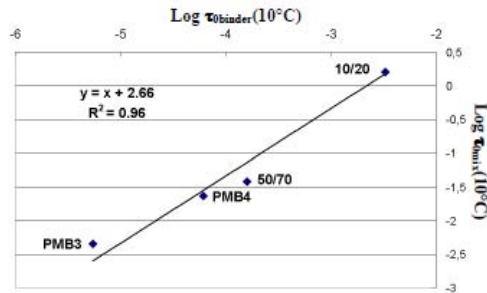


Figure 20. Relationship between  $\text{Log } \tau_{0\text{binder}}$  and  $\text{Log } \tau_{0\text{mix}}$  determined at  $10^\circ\text{C}$ .

Besides, as the  $a_T$  shift factors of bitumens and mixes are quasi-identical (previously evidenced by [BOU 95] [DIB 96] [BOU 96] [BOU 97] [OLA 03a] & [OLA 03b]), equation [6] brings about the following relation between  $\tau_{\text{binder}}$  and  $\tau_{\text{mix}}$  for the considered mix composition :

$$\tau_{\text{mix}}(\theta) = 10^\alpha * \tau_{\text{binder}}(\theta) \quad [16]$$

with  $\alpha = 2.66$  for any temperature " $\theta$ ".  $\alpha$  probably depends on the considered mix design and ageing during mixing as already mentioned before.

The previous developments of this chapter suppose that the Time-Temperature Superposition Principle ("TTSP") is verified, which was indeed observed as a first approximation over the considered range of temperature. A more general theory without considering the TTSP has been elaborated. But, considering the presented experimental data and the temperature range, the hypothesis of rheologically simple materials has been adopted.

#### 6. Prediction of mix modulus from binder one (no rheological model is considered)

A lot of work has already been done and presented in the literature to relate binder and mix moduli for a given mix design ([SHE 91], [LYT 93], [FRA 95], [BOU 96], [DIB 96], [FRA 96], [BOU 97], [GOA 00], [ZEN 01] & [OLA 03b]). Yet, these relationships are generally concerned with only the norm of the complex moduli ( $|G^*|$  and  $|E^*|$ ). In addition, some of these relationships are only valid over a rather narrow range of frequencies and temperatures, or even only for binder and

mix moduli measured in the same conditions of temperature and frequency. For instance, some simple linear relationships between  $|G^*|$  and  $|E^*|$  have been found in logarithmic scale, for a given mix design. However, for very different conditions of temperature and frequency, these relationships are not valid any more.

We present in this paper a relationship between the binder and the mix complex moduli (both the phase angle and the norm of the complex moduli are herein considered). Even though this relationship is constructed with the help of the "2S2P1D" model, it is independent of any rheological model.

From equations [13] and [14], the following relationship between the mix complex modulus and the binder one can be obtained :

$$E_{\text{mix}}^*(\omega, T) = E_{0_{\text{mix}}} + \left[ E_{\text{binder}}^*(10^\alpha \omega, T) - E_{0_{\text{binder}}} \right] \frac{E_{\infty_{\text{mix}}} - E_{0_{\text{mix}}}}{E_{\infty_{\text{binder}}} - E_{0_{\text{binder}}}} \quad [17]^3$$

If the binder complex modulus is known at a given temperature  $T$ , equation [17] gives the mix complex modulus at this temperature  $T$ . It introduces only three constants :  $E_{0_{\text{mix}}}$ ,  $E_{\infty_{\text{mix}}}$  and  $\alpha$ .

The two first constants have a clear physical meaning :  $E_{0_{\text{mix}}}$  and  $E_{\infty_{\text{mix}}}$  respectively correspond to the minimum and maximum asymptotic values of the norm of the complex modulus of mix, respectively at very low frequencies and high frequencies. As regards the additional parameter  $\alpha$  (equations [15] and [16]), it might depend on the considered mix design and/or ageing during mixing.

If the Time-Temperature Superposition Principle (TTSP) and thus equation [16] are verified for binders, the following property can be added (idem for mixes) :

$$E_{\text{binder}}^*(10^\alpha \omega, T) = E_{\text{binder}}^*(10^\alpha \omega a_T(T), T_s) \quad [18]$$

where  $a_T(T)$  : the shift factor at temperature  $T$ ,  
 $T_s$  the reference temperature.

Equation [17] is of the utmost importance since it consists of a very simple relationship between the mix complex modulus and the binder one for a given mix design and for any frequency-temperature combination. In fact, equation [17] corresponds to an isotropic expansion plus a translation of the binder curve in the Cole-Cole plane.

Moreover, the parameters of the introduced "2S2P1D" model do not appear in this formula [17]. This means that the transformation and relation between the mix complex modulus and the binder one do not depend on the considered model for binder. It can be applied to any model. Of course, "good" or "improved" models ("2S2P1D" model or CA model for instance) will give better simulations.

<sup>3</sup> Considering the experimental values in Table 3,  $E_{0_{\text{binder}}} < E_{\infty_{\text{binder}}}$  and  $E_{0_{\text{mix}}} < E_{\infty_{\text{mix}}}$ . Thus, equation [17] could be simplified in a first approximation.

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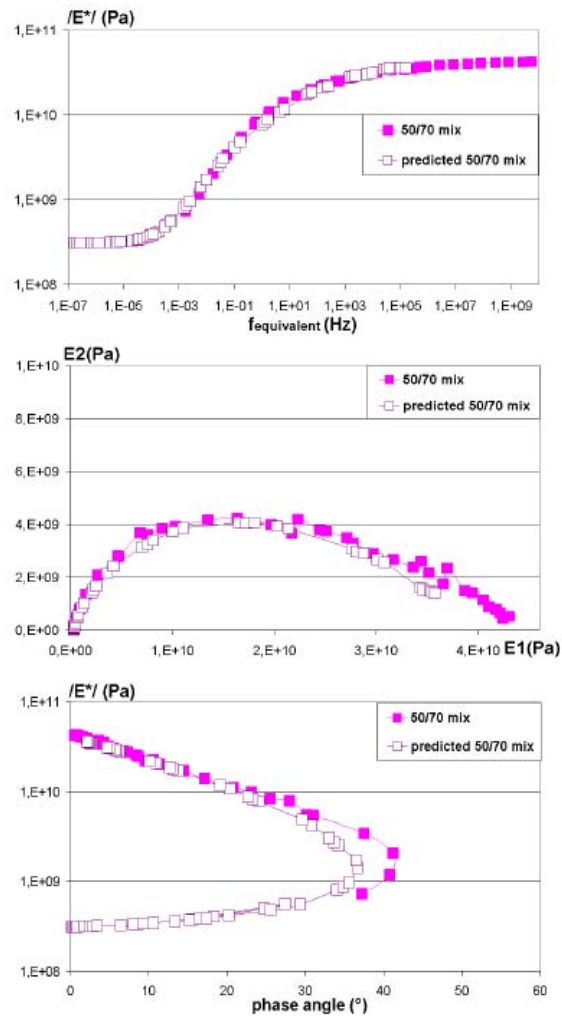
The efficiency of equation [17] is shown in Figure 21 (resp. Figure 22) which represents the comparison of the experimental data on the 50/70 (resp. PMB3) mix and the complex modulus predicted by equation [17], in which the experimental data on the 50/70 (resp. PMB3) bitumen are introduced. This result has also been checked with success for the two other mixtures made from the 10/20 and PMB4 binders, but, because of a lack of space, the corresponding curves are not presented herein.

No rheological model is needed for these comparisons, only the transformation given by equation [17], which has a geometrical explanation (isotropic expansion plus translation in the Cole-Cole plane), is considered.

Equation [17] appears as a powerful predicting tool for mix modulus prediction from binder modulus. For instance, the modulus of the five bituminous mixtures which have not been tested yet –made from 0/10, 20/30, 35/50, PMB1 and PMB2 binders– can be predicted from equation [17] since the glassy modulus of binders ( $E_{\infty\_binders} = 2\text{GPa}$  for the nine studied bitumens) and the static and glassy moduli of mixes for the considered mix design are known. If another mix composition is considered,  $E_{0\_mix}$  (resp.  $E_{\infty\_mix}$ ) could be obtained in a simple way by performing complex modulus test at  $+30^{\circ}\text{C}$  (resp.  $-30^{\circ}\text{C}$ ) over a range of frequencies in order to determine accurately the asymptotic value.

Consequently, tremendous mixture testing work could be saved, at least for the purpose of first estimates in the small strain domain (linear viscoelasticity).

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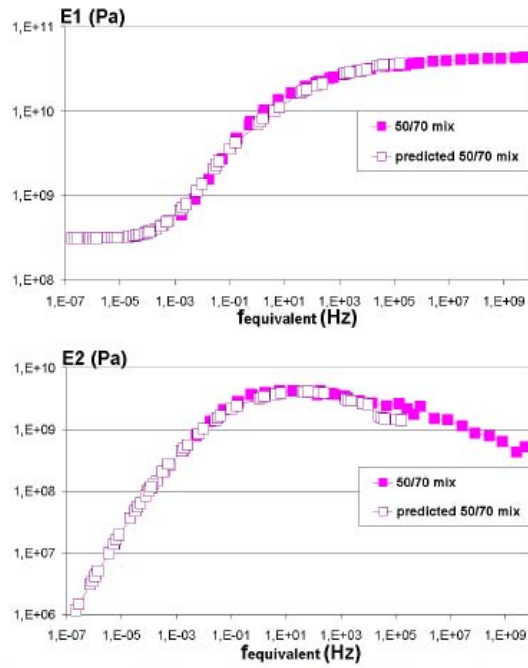
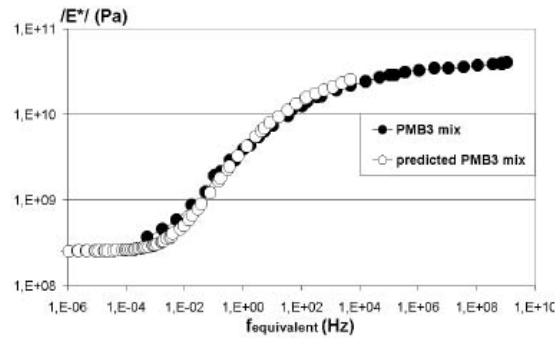
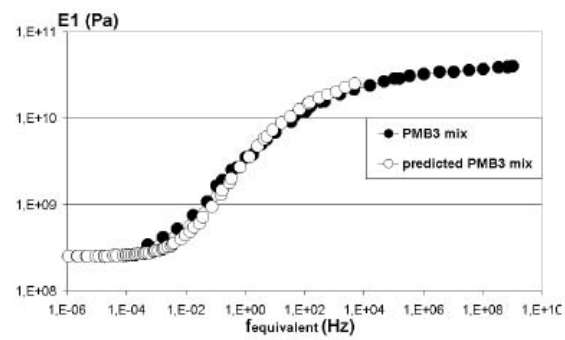
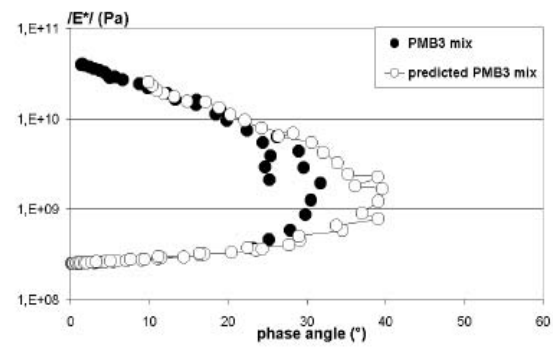
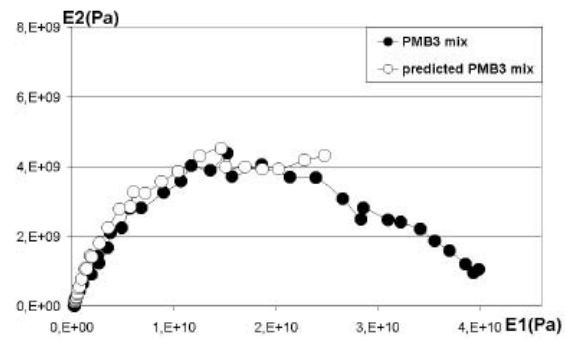


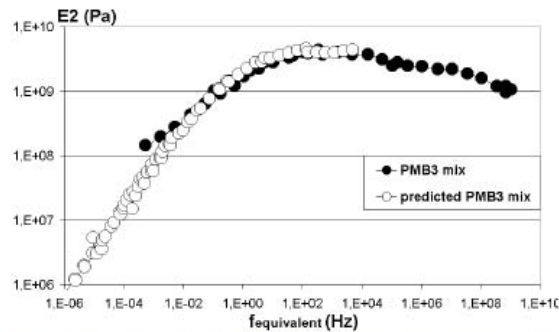
Figure 21. Comparison between the experimental data on the 50/70 mix and the complex modulus predicted from equation [17] using the experimental data on the 50/70 pen bitumen, at the reference temperature  $T_r=10^{\circ}\text{C}$ .



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**Figure 22.** Comparison between the experimental data on the PMB3 mix and the complex modulus predicted from equation [17] using the experimental data on the PMB3 binder, at the reference temperature  $T_r=10^\circ\text{C}$ .

## 7. Conclusions

A rational approach which consists in comparing the properties of binders and mixes only in the small strain amplitude domain (where Linear ViscoElastic behavior is observed), was considered in this paper. The aim of this work was to seek both handy and pertinent relations between the rheological properties of binders and mixes in the linear viscoelastic domain from the use of a pertinent rheological modeling based on physical analogical elements. The viscoelastic behavior of bituminous binders and mixes was evaluated using complex modulus tests. The following conclusions may be drawn.

The construction of the polymer modified binders and mixes master curves relies on the introduced "Partial Time-Temperature Superposition Principle" (PTTSP). The PTTSP is herein considered as an effective approximate approach for analyzing viscoelastic data in the case of PMB's.

An alternative general model "2S2P1D" (generalization of the Huet-Sayegh model) valid for both the bituminous binders and mixtures and based on a simple combination of physical elements (spring, dashpot and parabolic element) has been proposed. The Huet-Sayegh model [SAY 65] has been adapted by adding a linear dashpot in series with two parabolic elements and the spring of rigidity  $E_\infty-E_0$  (Figure 16). Thus, in particular for binders at low frequency, the model is equivalent to a linear dashpot (Newtonian viscous material). Like the Huet-Sayegh model, the introduced model has a continuous spectrum (*i.e.* can be represented by an infinity of Kelvin-Voigt elements in series or Maxwell elements in parallel).

This comprehensive model translates correctly the linear viscoelastic behavior of nine very different binders and four mixes (with one mix design) for any range of

frequencies and temperatures. As only one aggregate formula has been studied in this paper, this point remains to be checked with other mix compositions.

The use of the introduced general model "2S2P1D" (equation [11]) confirmed that the rheology of bituminous mixes can be correlated to that of binders. In fact, 4 out of 7 constants, needed at a given temperature, are identical for the binder and the respective mix.

In addition, for the studied mix design, a new and simple relation (equation [17]) has been proposed to relate the complex moduli of binders and mixes. Moreover, the parameters of the introduced general model do not appear in this formula [17]. This means that the relation between the mix complex modulus and the binder one may be intrinsic. It does not depend on any considered model for binder and can be applied to any "good" or "improved" model ("2S2P1D" model or another model) or more directly from the experimental data on binder. Only three parameters  $\alpha$ ,  $E_{0\_mix}$  and  $E_{\infty\_mix}$  need to be determined. From the literature,  $E_{0\_mix}$  and  $E_{\infty\_mix}$  highly depend on mix composition. As regards the additional parameter  $\alpha$  (equations [15] and [16]), it might depend on the considered mix design and/or ageing during mixing.

It has to be underlined that equation [17] does not suppose the Time-Temperature Superposition Principle. If this property holds for the binder, it will appear as automatically verified for the corresponding mix.

Finally, this powerful predicting tool could be used for any given binder, provided that the coefficient  $\alpha$  and the static and glassy moduli of the considered mix design are known (equations [15] & [17]). Briefly speaking, if a bitumen  $i$  and the corresponding  $i$  mix –for a given mix design– have been tested, the complex modulus of a bitumen  $j$  could be then sufficient to predict the complex modulus of the corresponding  $j$  mix, for the considered mix design. This result is of prime importance as consequently tremendous mixture testing work could be saved. Meanwhile, elegant as the basic idea may seem, formula [17] still needs to be validated with other aggregate skeleton and binder content.

Most of the presented results suppose that the linear viscoelastic material is thermo-rheologically simple. A more general theory which does not respect this hypothesis had been developed and can be used if experimental data reveals a more complex behavior for the considered bituminous materials.

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#### Acknowledgements

The authors are thankful for the support that was provided by TotalFinaElf by furnishing the five pure bitumens and the cross-linked binder, and by performing the complex modulus tests on the studied bitumens.

Furthermore, the authors gratefully acknowledge the great help provided by the staffs of both the Appia and Eurovia laboratories, in particular for the study of the four bituminous mixtures and also by furnishing three polymer modified bitumens.

Finally the staffs of Appia, Eurovia and TotalFinaElf have all contributed to valuable and fruitful discussions during the course of this work.

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## Annex 1

Material	C <sub>1</sub>	C <sub>2</sub>
0/10 binder	20	142
10/20 binder	23	170
20/30 binder	23	171
35/50 binder	19	147
50/70 binder	22	164
PMB1 binder	19	149
PMB2 binder	19	153
PMB3 binder	17	149
PMB4 binder	18	138
10/20 mix	17	134
50/70 mix	20	135
PMB3 mix	20	135
PMB4 mix	20	145

**Table 4.** WLF coefficients of the nine bitumens and the four studied mixtures, for the considered reference temperature  $T_s=10^\circ\text{C}$  (cf. WLF equation [7]).



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## Annex 2

## 1. The Maxwell model

Figure 23 represents a Maxwell element. Equations [19] and [20] give respectively the creep function  $J(t)$  and the relaxation function  $R(t)$  for this model.

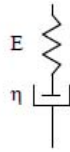


Figure 23. Maxwell element.

$$J(t) = \frac{1}{E} + \frac{t}{\eta} \quad [19]$$

$$R(t) = E e^{-\frac{t}{\tau}} \quad [20]$$

with  $\tau = \frac{\eta}{E}$ , where  $\tau$  is the relaxation time.

If  $\tau$  is a function of temperature, the Time-Temperature Superposition Principle ("TTSP") is valid. This remark can be applied to the next paragraphs as well.

The complex modulus of the Maxwell element is as follows :

$$E^*(\omega) = E \frac{i\omega\tau}{1+i\omega\tau} = \frac{E \omega^2 \eta^2 + i E^2 \omega \eta}{E^2 + \omega^2 \eta^2} \quad [21]$$

where  $i$  : complex number defined by  $i^2 = -1$ .

## 2. The Kelvin-Voigt model

Figure 24 represents a Kelvin-Voigt element. Equations [22] and [23] give respectively the creep function  $J(t)$  and the relaxation function  $R(t)$  for this model.

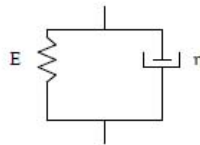


Figure 24. Kelvin-Voigt element.

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$$J(t) = \frac{1}{E} (1 - e^{-\frac{t}{\tau}}) \text{ with } \tau = \frac{\eta}{E} \quad [22]$$

$$R(t) = E + \eta \delta(t), \text{ with } \delta \text{ Dirac function} \quad [23]$$

The complex modulus of the Maxwell element is as follows :

$$E^*(\omega) = E + i\omega\eta \quad [24]$$

### 3. Examples of the generalized Maxwell model and Kelvin-Voigt models

#### 3.1. Generalized Maxwell model

A group of Maxwell elements in parallel represents a discrete spectrum of relaxation times, each time  $\tau_i$  being associated with a spectral strength  $E_i$  ( $\tau_i = \eta_i/E_i$ ). A spring may also be placed in parallel to the group of the  $n$  Maxwell elements (Figure 8). Then, the relaxation modulus of this model can be written as follows [FER 80] :

$$R(t) = E_\infty + \sum_{i=1}^n E_i e^{-\frac{t}{\tau_i}} \quad [25]$$

The corresponding complex modulus  $E^*$  has the following form :

$$E^*(i\omega) = E_\infty + \sum_{i=1}^n E_i \frac{i\omega\tau_i}{1 + i\omega\tau_i} \quad [26]$$

The model which corresponds to equations [25] and [26] is characterized by a "discrete relaxation spectrum", *i.e.* constituted by a discrete number of elements, each element having a relaxation time  $\tau_i$  associated with a spectral strength  $E_i$ .

If the number  $n$  of Maxwell elements is increased without limit, the discrete spectrum becomes a continuous curve, and consequently the modulus becomes a continuous function  $E(\tau)$  of relaxation times.

Therefore, the relaxation function of such a model can be written :

$$R(t) = E_\infty + \int_{\ln(\tau)=-\infty}^{\ln(\tau)=+\infty} H(\tau) e^{-\frac{t}{\tau}} d \ln \tau \quad [27]$$

$H(\tau) d \ln(\tau)$  can be considered as the modulus associated with relaxation times whose logarithms lie in the range between  $\ln(\tau)$  and  $\ln(\tau)+d \ln(\tau)$ .

In the frequency domain, the complex modulus  $E^*(\omega, T)$  can be written :

$$E^*(\omega, T) = \int_{\ln(\tau)=-\infty}^{\ln(\tau)=+\infty} H(\tau) \frac{i\omega\tau}{1 + i\omega\tau} d \ln \tau \quad [28]$$

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Any linear viscoelastic material can be modeled with this formalism [MAN 66].

### 3.2. Generalized Kelvin-Voigt model

The creep function and the complex modulus of this model (Figure 9) are respectively as follows :

$$J(t) = \sum_{j=1}^n \frac{1}{E_j} \left( 1 - e^{-\frac{t}{\tau_j}} \right) + \frac{1}{E_\infty} + \frac{1}{\eta_0} \quad [29]$$

$$E^*(i\omega) = \left( \sum_{j=1}^n \frac{1}{E_j + i\eta_j\omega} + \frac{1}{E_\infty} + \frac{1}{i\eta_0\omega} \right)^{-1} \quad [30]$$

with  $\tau_j = \frac{\eta_j}{E_j}$ .

If  $n$  tends to infinity, a continuous spectrum can also be introduced.

### 3.3. General consideration

The generalized Maxwell or Kelvin-Voigt models with a discrete number of elements ( $n$ ) are not always appropriate for the description of a complex linear viscoelastic behavior. Nonetheless, when increasing  $n$ , the modeled behavior is enhanced. On the opposite, an infinite number of Maxwell or Kelvin-Voigt models characterized with a continuous spectrum is always appropriate.

More generally, one should bear in mind that any linear viscoelastic material can be represented by an assembly, eventually of an infinite number, of Maxwell elements or Kelvin-Voigt elements respectively in parallel or in series [MAN 66].

## 4. The Huet model

The complex modulus can be written :

$$E^*(\omega) = \frac{E_\infty}{1 + \delta (i\omega\tau)^{-k} + (i\omega\tau)^{-h}} \quad [31]$$

with  $i$  : complex number defined by  $i^2 = -1$ ,

$\omega = 2\pi$ \*frequency, the solicitation pulsation,

$E_\infty$  : limit of complex modulus when  $\omega\tau \rightarrow \infty$ ,

$h, k$  : exponents such as  $1 > h > k > 0$ ,

$\delta$  : dimensionless constant.

$\tau$  : characteristic time, which value varies only with temperature; accounts for the Time Temperature Superposition Principle (TTSP).

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No analytical expression of the relaxation function  $R(t)$  is available. The creep function is given by equation [32] :

$$J(t) = \frac{1}{E_{\infty}} \left( 1 + \delta \frac{(t/\tau)^k}{\Gamma(k+1)} + \delta \frac{(t/\tau)^h}{\Gamma(h+1)} \right) \quad [32]$$

## Annex 3

As already underlined by Christensen and Anderson, the CA model [CHR 92] does not fit the experimental results well at high temperatures and low frequencies (cf. Black diagram, Figure 25). Likewise, at low temperatures and high frequencies, as shown in Figure 25 (Cole-Cole plane), the fitting is not so good any more. From our results on the nine tested bitumens, the mathematical formula [1a] seems only suitable to approximate the norm of the stiffness complex modulus  $|E^*|$ .

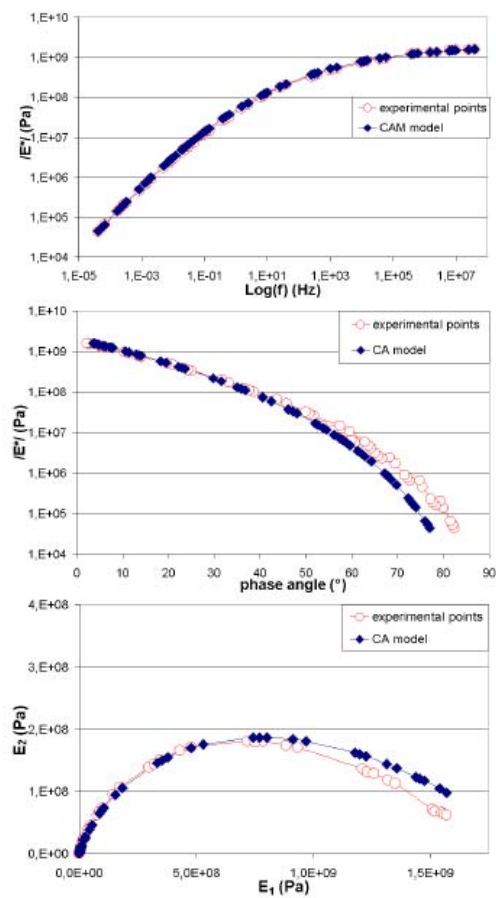
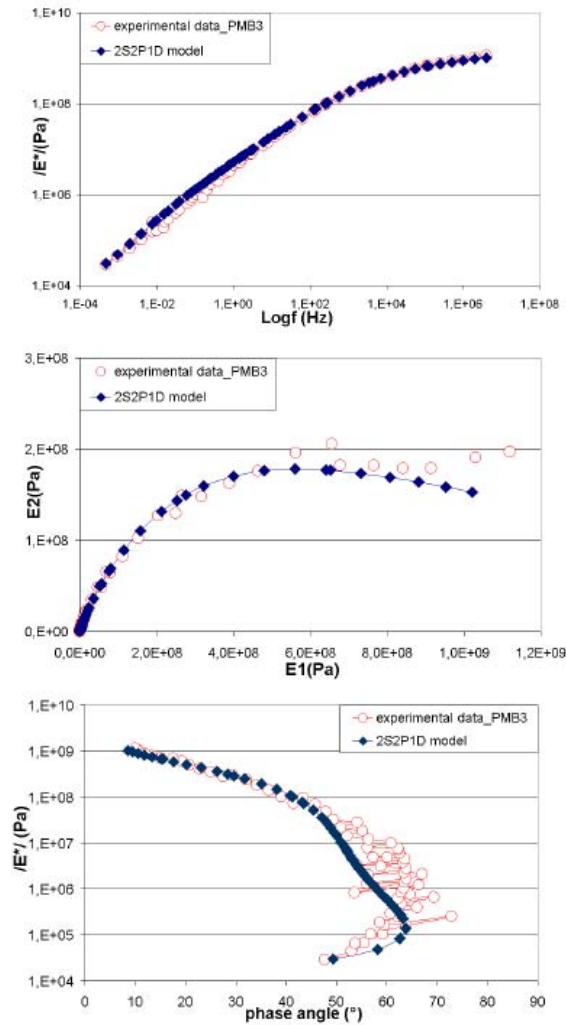


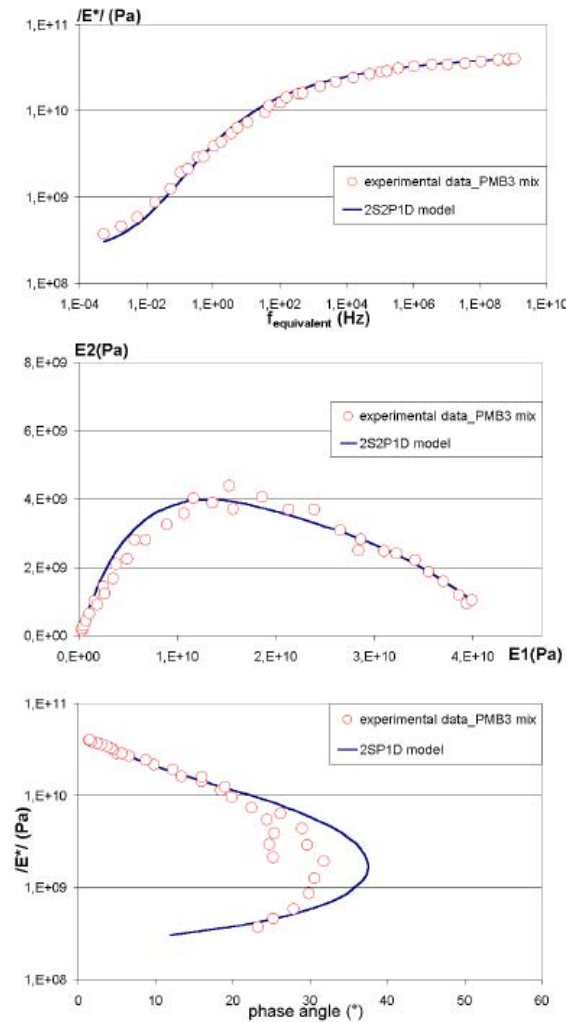
Figure 25. Comparison between experimental results on the 50/70 pen bitumen and the CA model [CHR 92], equations [1a] and [1b].

## Annex 4



**Figure 26.** Comparison between the experimental complex modulus data on the PMB3 binder and the corresponding calibrated "2S2P1D" model ( $T_2=10^\circ\text{C}$ ). Constants are given in Table 3.

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**Figure 27.** Comparison between the experimental complex modulus data on the PMB3 mixture and the corresponding calibrated "2S2P1D" model ( $T_2=10^\circ\text{C}$ ). Constants are given in Table 3.

## **THE “DBN” MODEL : A THERMO-VISCO-ELASTO-PLASTIC APPROACH FOR PAVEMENT BEHAVIOR MODELING. APPLICATION TO DIRECT TENSION TEST AND THERMAL STRESS RESTRAINED SPECIMEN TEST**

**François Olard and Hervé Di Benedetto**

### **Abstract**

Mix behavior is complex. Following applied loading, very different types of properties can be observed, such as linear viscoelasticity (LVE) for very small strain amplitudes, non linearity for higher strains, fatigue or rutting for a great number of cycles, brittle or ductile failure, thixotropy and healing. These different “typical” kinds of behavior are introduced. Then, the proposed “DBN” (Di Benedetto-Neifar) law is presented. This law is very versatile and provides an effective means of describing the different types of mix behavior by using the same formalism. An emphasis is especially placed on explaining how the law can cope with different “typical” mix behaviors. The complexity of the description and the number of constants can be very different following desired sophistication for the modeling. In particular, some comparisons between experimental DTT and TSRST results and numerical simulations based on the DBN law are presented. Some of the presented aspects still need further research work.

**Key Words:** Bituminous Mixes, Rheological Law, Modeling, Experimental Characterization, Complex Modulus, Direct Tension Test, Thermal Stress Restrained Specimen Test

### **Introduction**

Roadway structures are subjected to complex phenomena under the effect of external solicitations. Mechanical, thermal, physical and chemical phenomena, for example, can be observed. Moreover, a coupling between these effects often appear.

Faced with the complexity of the observed problems, developed design methods are often semi-empirical. These methods are all the more limited as new types of structures and more powerful materials are used and as the traffic is in constant growth. The use of more rational methods thus seems a needed evolution. Work has already been done in this direction, like, for example, the developments of the French method [1], applied since the 70's, and, more recently, efforts engaged by the US within the framework of the Strategic Highway Research Program (1988-93, continued after 1996) which allowed the development of the Superpave method (Superpave, 1996-97) and the design guide AASHTO 2002 (AASHTO Design Guides). However, there is still a gap between the results obtained from the research on the behavior of bituminous materials and the introduction of this knowledge in design methods. Moreover, the development of an accurate and advanced rational rheological law for bituminous mixes is still necessary for realistic performance prediction of bituminous pavements (see e.g. project NCSHRP 9-19 [2]).

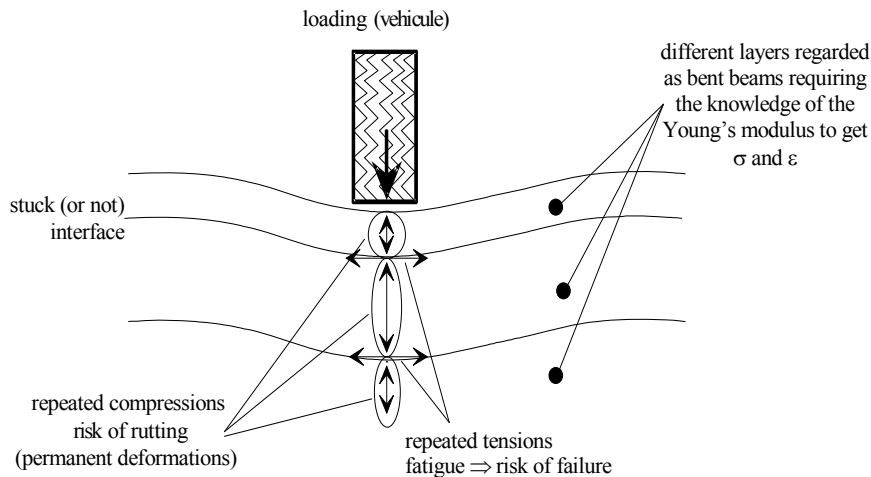
First of all, the solicitations to which the bituminous roadways are subjected are presented. From the analysis of these solicitations, different aspects of the thermo-mechanical behavior of bituminous mixes are then considered. The description of these different kinds of behavior is introduced in the following paragraphs. The recent results analyzed in this paper, show that some improvements are still possible, even necessary, in the design methods of road structures.



## Background

### *Solicitations in bituminous roadways*

*General* The two main solicitations are related to the passage of vehicles (traffic effect) and to the effects of climatic changes, mainly because of the temperature variations (thermal effects). These two main solicitations are far more important than the other types of solicitations such as degradations related to the moisture or the ageing of materials.



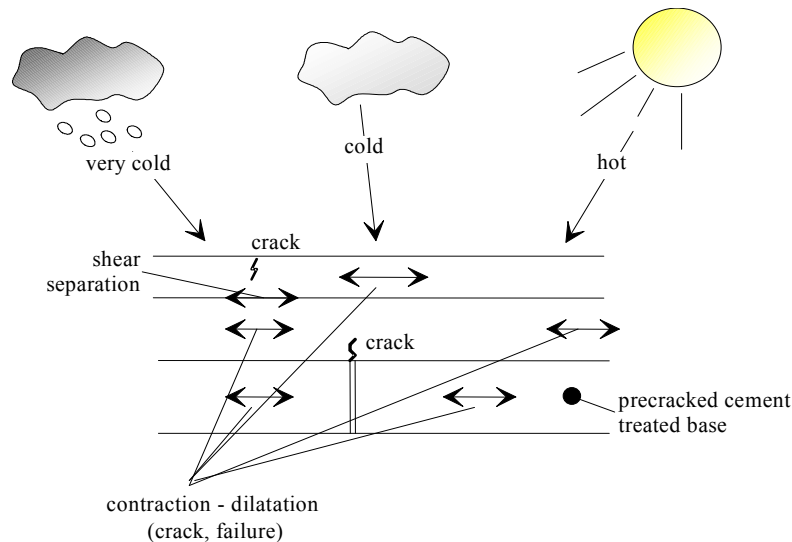
**Figure 1. Schematic of the traffic induced solicitations.**

*Traffic influence* Each roadway layer is subjected to a complex bending and compression state due to the effect of the traffic (Figure 1). The amplitudes of cyclic loadings remain low (deformation of the order of 10-4m/m) which can explain why the calculation of strains and stresses, for each cycle, is traditionally made by considering an isotropic linear elastic multi-layer model. This requires, initially, the determination of values of Young's modulus and Poisson's ratio.

On account of the particular properties brought by the bitumen, the bituminous mixes have a rate-dependant behavior (thus a rate-dependant modulus) (and also a temperature dependant behavior as underlined in the next paragraph). The assumption of an elastic behavior thus corresponds to an approximation which can turn out to be a very rough hypothesis. In particular, the effects of non-linearities, damage and irreversibilities accumulate with the number of cycles which can reach several millions in a roadway life. This creates fatigue and rutting in the pavement.

*Temperature influence* The temperature has two main mechanical effects :

- A stiffness (modulus) change of the material. A bituminous mixture becomes “softer” at higher temperatures. More generally, bituminous mixtures have a complex temperature-sensitive behavior, that is to say that their viscoplastic behavior changes according to the considered temperature.
- The development of stresses and deformations within the material because of the thermal dilatations and contractions during the temperature changes (Figure 2).



**Figure 2. Schematic of the temperature induced solicitations.**

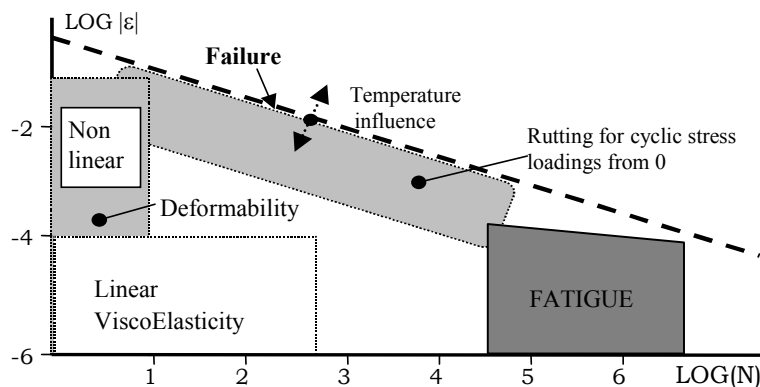
### ***Behavior of bituminous materials in pavement structures***

Apart from the very thin surface layers, the different bituminous layers have a structural effect. In order to characterize this effect and its evolution with time, mechanical properties of mixes have to be modeled considering the following aspects :

- Stiffness and stiffness evolution with time.
- Fatigue and damage law evolution.
- Accumulation of permanent deformations.
- Crack initiation and crack propagation, in particular at low temperatures.

These four properties are of primary importance when modeling road behavior. The first one, observed for very small strains, corresponds to the linear viscoelastic behavior of mixes. From analytical or numerical methods, the linear viscoelastic law allows to obtain the stress and strain fields within the pavement. The three other aspects are respectively at the origin of major distresses : degradation by fatigue, rutting and crack propagation. Each of these properties or distresses appears for a given domain of loading and corresponds to a specific type of mix behavior.

Figure 3 highlights the domains corresponding to the different aspects introduced before and the corresponding “typical” types of mix behavior according to the strain amplitude and the number of applied cyclic loadings (Di Benedetto [3], Olard [4]).



**Figure 3. Typical bituminous mix behavior domains, ( $|\epsilon|$  strain amplitude – N number of cycles).**

### Presentation of the “DBN law” (Di Benedetto-Neifar)

We propose hereafter a general constitutive law, called “DBN law”, which describes, with one formalism, the different kinds of mix behavior following the considered loading domain (Figure 3). The framework is different from that of the rheological laws already proposed for bituminous mixtures (e.g. 5, 6, 7, 8, 9).

The basic formulation of the law can be simplified or adapted according to the studied properties. The law can then be very simple and easy to use (linear viscoelastic or even elastic) or more complicated (introduction of non linearity, permanent deformation, or fatigue, etc.). Considering the thermo-sensitivity of bituminous materials, the temperature influence is always considered.

First, a unidimensional formulation is presented. We then explain how non linearity, brittle and flow failure, permanent deformation and fatigue can be modeled with the “DBN” law. Using the same formalism, the introduction of thixotropy and healing, which sometimes seem to play an important role, is also possible. Finally, a rapid explanation of some bases for the tri-dimensional case is given.

Due to the lack of space, only a rapid description is given hereafter. For more details on the general formulation of the law, the reader is referred to the following references : e.g. Neifar [10], Neifar & Di Benedetto [11], Olard [4] [12]. The generalized analogical body presenting the law is drawn in Figure 4.a. The “EPI” type bodies are non viscous models and translate any non viscous behavior (i.e. rate-independent response). The “Vi” type bodies are purely viscous models and introduce the viscous irreversibility and the thermal sensitivity.

As is underlined further, the number “n” of elementary bodies can be arbitrarily chosen. This does not change the number of constants for the model. If n is high, the calibration can be closer to the experimental data, but the simulation calculus of complex stress path loadings becomes more time consuming. An extension of the law for n tending toward infinity has also been developed (discrete spectrum to continuous spectrum (Neifar [10], Neifar & Di Benedetto [11], Di Benedetto et al. [14])). This extension does not increase the number of needed constants for the model calibration.

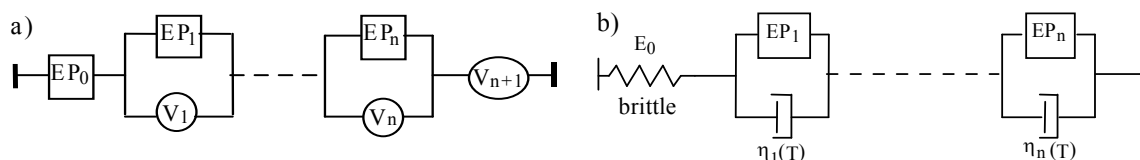


Figure 4. (a) Generalized analogical model (Di Benedetto 1987 [13]); (b) DBN model for bituminous mixtures.

#### Description of the EPI body behavior

Each EPI element translates a behavior qualitatively identical to that of granular non cohesive materials. The response of this body is independent of the temperature and the loading rate (non viscous material). The incremental formalism of “interpolation” type, developed by Di Benedetto [15] for sands, can thus be directly used to qualify EPI bodies. For the considered unidimensional case, the behavior of the body is described by a relation between the stress increase ( $\Delta\sigma$ ) and strain increase ( $\Delta\varepsilon$ ) starting from the last point of loading reversal :

$$\Delta\sigma = f(\Delta\varepsilon) \quad (1)$$

The function  $f$  can be obtained from the 2 virgin curves, “ $f^+$ ” with an asymptote “ $s^+$ ” for loading case, and “ $f^-$ ” with an asymptote “ $s^-$ ” for unloading case. A cyclic rule has to be introduced. Figure 5 gives an example of a simple cyclic rule, which is a generalization of the Masing rule.

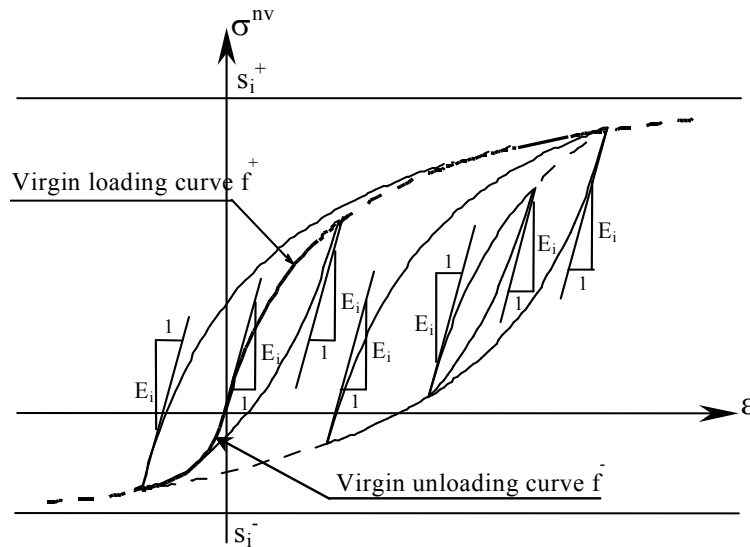


Figure 5. EPI behavior : example of cycling sequences.

In the proposed simulations, “ $f$ ” is hyperbolic and only 3 constants ( $s_i^+$ ,  $s_i^-$  and  $E_i$ ) are needed for each EPI body (cf. Figure 5), except for EP0 which is a spring of rigidity  $E_0$  up to a brittle limit  $\sigma_c(T)$  as explained further :

- $E_i$  is the slope at the origin.
- $s_i^+$  is the asymptote of plasticity in compression (flow in compression) and  $s_i^-$  represents the asymptote of plasticity in tension (flow in tension).

The equation of the virgin curve  $f^+$  is as follows :

$$\sigma_i = \frac{E_i \varepsilon_i}{1 + \frac{E_i \varepsilon_i}{s_i^+}} = f^+(\varepsilon_i) \quad (2)$$

As regards the equation of the virgin curve  $f^-$ ,  $s_i^+$  must be replaced by  $s_i^-$  in equation (2). Besides, for very small cycles, the EPI body behavior is equivalent to that of a spring of rigidity  $E_i$ .

The definition of a viscoplastic failure criterion ( $\sigma_{\text{failure}} = F(\dot{\varepsilon})$ ) makes it possible to calculate the values of  $s_i^+$  and  $s_i^-$ . Through tension/compression tests at constant strain rate, the ratio “ $k$ ” between  $s_i^+$  and  $s_i^-$  can be considered as the same for each EPI body (Di Benedetto et Yan [16]). The expression  $f^-$  of the virgin unloading curve can be deduced from  $f^+$  by using the following expression ( $-k = s_i^+ / s_i^-$  for every  $i=1$  to  $n$ ) :

$$f^-(\varepsilon_i) = -kf^+\left(-\frac{\varepsilon_i}{k}\right) = \sigma_i \quad (2\text{bis})$$

In addition, considering the extension of the Masing rule, the loading or unloading branches, which are not on the virgin curve (Figure 5) can be expressed by:

$$\Delta\sigma_i = f(\Delta\varepsilon_i) = (\text{sgn}) (1+k) f^+\left(\text{sgn} \frac{\Delta\varepsilon_i}{1+k}\right) \quad (3)$$

where  $\text{sgn}$  is the sign of loading ( $i=1$  for loading and  $i=-1$  for unloading).

**Description of the Vi body behavior**

The considered form for bituminous mixtures suppose that the “Vi” bodies are linear dashpot characterized by their viscosity  $\eta_i(T)$  which is a function of the temperature T. As it can be shown theoretically, only  $\eta_i$  depends on T, to take into account the Time-Temperature Superposition Principle (TTSP). Then the unidimensional model used for the simulations of mix behavior has the structure presented as shown in Figure 4.b. The viscous element  $V_{n+1}$  normally in series is not present, for the bituminous mixes are considered as solid type bodies.

**Brittle behavior of the spring of rigidity E0**

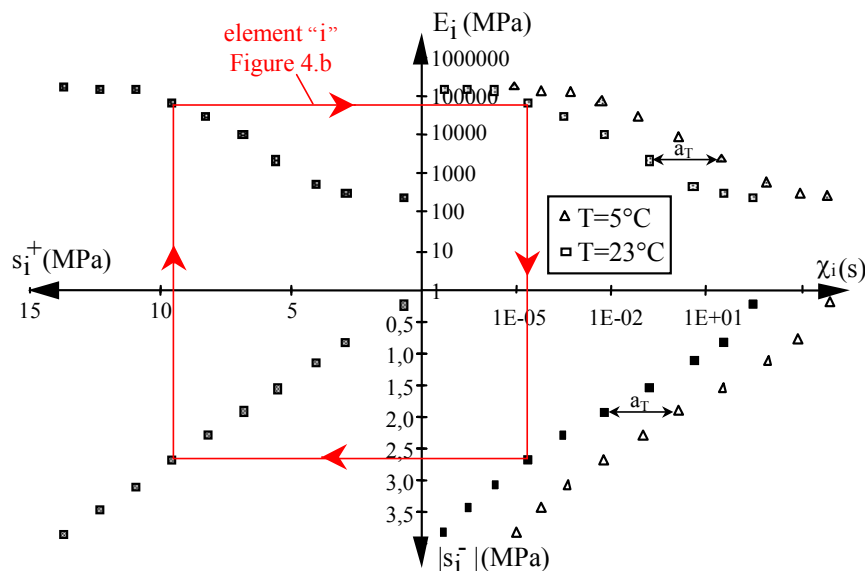
The spring of rigidity  $E_0$  gives the instantaneous deformations of the mix. It also corresponds to the limit of the complex modulus ( $E^*$ ) obtained for very high frequencies. In addition, a failure element is integrated into this spring in order to model the brittle failure of the mix. This failure element brakes as soon as a failure tensile stress is reached. This failure tensile stress is obtained experimentally from the tensile strength of the mix. It increases monotonically with the temperature (Olard [4], Olard et al. [17] [18] [19]). More details are given further in the section “brittle failure of the spring of rigidity  $E_0$ ”.

**Spectral representation of the model**

Figure 6 shows a representation of the generalized model in the discrete case with 15 elements by the discrete spectra. The DBN model can be widened by using continuous spectra. This extension makes it possible to be freed from the discrete aspect. An “infinite” number of elements  $E_{Pi}$  and  $V_i$  can be considered. A spectrum for the very small deformations and another for viscoplastic flow are introduced (Neifar [10]).

The generalized model is defined from the viscoelastic spectrum, the spectrum of the thresholds of plasticity, and from the Time-Temperature Superposition Principle (TTSP).

More information is given on the spectral representation of the DBN model by Neifar [10].



**Figure 6. Example of representation of the 10-element DBN model by the discrete spectra.  $a_T$  is the shift factor (equation (6)) and  $\chi_i = E_i/\eta_i$ . Four data points characterize an element “i” (dashpot “i” and  $E_{Pi}$ ) of the DBN model (Figure 4.b).**

## How the same formalism can be adapted to describe the different “typical” kinds of mix behavior

As shown in Figure 3, the bituminous mixes present different “typical” kinds of behavior following the considered domain of loading. These different types of behavior can be described by the general formalism of the DBN law (Figure 4), but the  $E_i$  and  $V_i$  behaviors need to be explained and calibrated. It is clear that the more general the description, the more complex the rheological function of each body. It is also of natural practical sense to consider, for a given property, only the simplified form of the model that allows to model that property. Considering these two remarks, we propose different calibration processes, in the next paragraphs. Each one is adapted to a considered domain of behavior (Figure 3) and takes into account the experimentally observed “typical” type of behavior. For a considered kind of behavior, only the obtained asymptotic expression of the law can be considered. It will give the same result as the general formulation by using fewer constants and simpler equations. The law is then very versatile and depending on required accuracy, simple or more complex equations can be considered. The evolution from one type of behavior to another remains continuous (no “switch” boundary) when considering the general expression of the law.

### Current developments for the DBN law

#### *Calibration in the small strain domain and for a small number of cycles : linear viscoelastic (LVE) behavior*

The small strain domain corresponds to the domain where the mix behavior can be considered as linear. Charif [20], Doubbaneh [21] and Airey et al. [22] [23] have experimentally found the limit of the linear domain around some  $10^{-5}$  to  $10^{-4}$  m/m for bituminous mixtures. A value of  $10^{-4}$  below which the behavior is linear can reasonably be considered.

In the small strain domain (strain amplitude  $< 10^{-4}$ ) and for a small number of cycles, the mix behavior is linear viscoelastic and respect the Time-Temperature Superposition Principle (TTSP). Then, the calibration of the law using experimental data from that domain gives the value of  $E_i$  (Figure 5) and  $\eta_i$  (Figure 4.b).

In this domain, the  $E_i$  elements have a linear elastic behavior and can be replaced by springs of rigidity  $E_i$ . The selected discrete model becomes an assembly in series of Kelvin-Voigt analogical bodies, whose complex modulus is given by :

$$E^{*DBN}(j\omega, T) = \left( \frac{1}{E_0} + \sum_{i=1}^{10} \frac{1}{E_i + j\omega\eta_i(T)} \right)^{-1} \quad (4)$$

where :

- $j$  = the complex number:  $j^2 = -1$  ;
- $\omega$  = the pulsation ( $\omega = 2\pi f$ ,  $f$  is the frequency) ;
- $T$  = the temperature.

The obtained asymptotic body is represented in Figure 7.a.

Extensive work on the modeling of complex modulus of bituminous binders and mixes allows the two authors to formulate a general linear viscoelastic law with a continuum spectrum called 2S2P1D (2 Springs, 2 Parabolic elements, 1 Dashpot, cf. Figure 7.b). The 2S2P1D model consists of a generalization of the Huet-Sayegh model (see e.g. Olard [4], Olard & Di Benedetto [24], Di Benedetto et al. [25]). Due to a lack of place, details of this

work are not presented here but the reader can refer to the three previous references. It has been shown in these three papers that the 2S2P1D model is a powerful tool to translate the linear viscoelastic behavior (i.e. in the small strain domain) of bituminous binders, mastics and mixes, for a very wide range of frequency and temperature. It only needs 7 constants at a given temperature.

The calibration process of the DBN model then corresponds to an optimization in the frequency domain with the results obtained from the 2S2P1D model. The great interest of this procedure is to chose any number of elements for the DBN model once the 7 constants of the 2S2P1D model are fitted from experimental data.

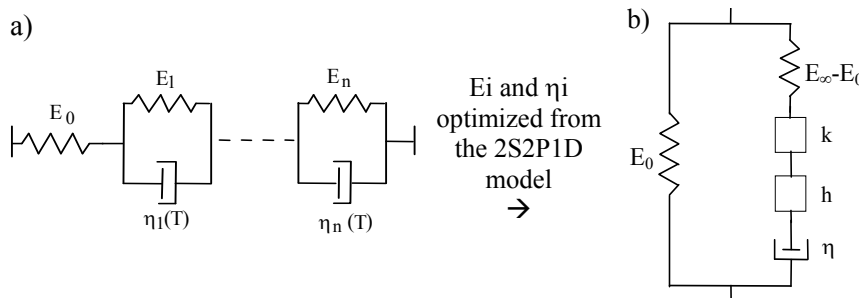
The complex modulus of the 2S2P1D model has the following expression :

$$E^{*2S2P1D}(j\omega\tau) = E(0) + \frac{E(\infty) - E(0)}{1 + \delta(j\omega\tau)^{-k} + (j\omega\tau)^{-h} + (j\omega\beta\tau)^{-1}} \quad (5)$$

$\tau$  is a function of the temperature (T) which allows to take into account the Time-Temperature Superposition Principle (TTSP) :

$$\tau = \tau_0 a_T(T) = \tau_0 10^{-\frac{C_1(T-T_s)}{C_2+T-T_s}} \quad (6)$$

where  $\tau_0$  is a constant to be determined at the –arbitrarily chosen– reference temperature  $T_s$ , and  $a_T$  corresponds to the shift factor given by William, Landel and Ferry [26] ( $C_1$  and  $C_2$  are two constants to be determined).



**Figure 7. (a) Analogical asymptotic form of the law in the LVE domain; (b) 2S2P1D model (Olard & Di Benedetto [24]).**

The  $2n+1$  parameters ( $E_i, \eta_i$ ) of the discrete DBN model (cf. Figure 7.a where  $E_{Pi}$  is a spring of rigidity  $E_i$ ) are obtained from an optimization process using only the 7 constants of the 2S2P1D model and the 3 constants of the WLF equation (Ferry [26]) whatever the number of elements “n”. This optimization process consists in minimizing, at the reference temperature ( $T_s$ ), the sum of the distances between the complex modulus of the two models (2S2P1D model and “DBN” model) at  $N$  points of pulsation  $\omega_i$ . The minimization of the following expression is made by using the *Solver* feature of MS Excel :

$$\sum_{i=1}^{i=N} \left( [E_1^{2S2P1D}(\omega_i) - E_1^{DBN}(\omega_i)]^2 + [E_2^{2S2P1D}(\omega_i) - E_2^{DBN}(\omega_i)]^2 \right) \quad (7)$$

$E12S2P1D$  et  $E22S2P1D$  are respectively the real part and the imaginary part of the complex modulus  $E^{*2S2P1D}$  given by equation (5).  $E1DBN$  et  $E2DBN$  are respectively the real part and the imaginary part of the complex modulus  $E^{*DBN}$  given by equation (4).

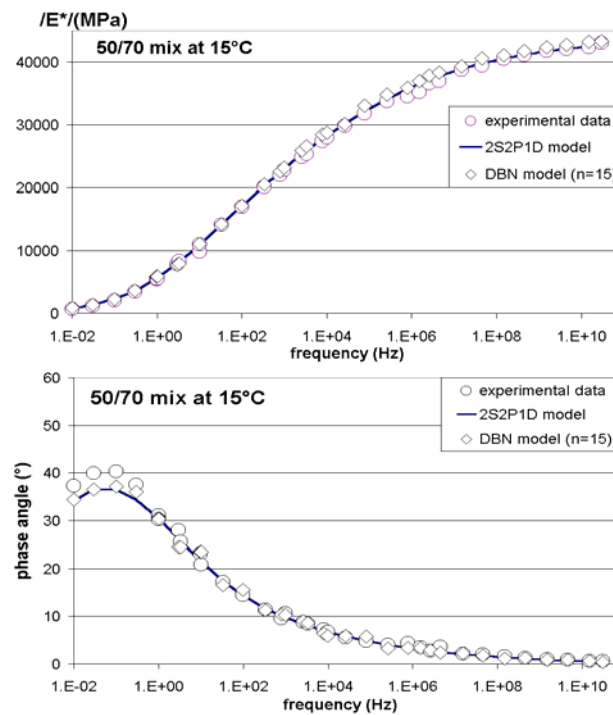
The minimization of expression (7) is carried out by using a succession of monotonic relaxation times  $\tau_i$ . The two following conditions are imposed :

- the suite  $(E_j)_{j \in \{1, \dots, 15\}}$  is increasing,
- the suite  $(\eta_j)_{j \in \{1, \dots, 15\}}$  is decreasing.

Note that the “glassy modulus”  $E(\infty)$  in equation (5) corresponds to the rigidity  $E_0$  of the discrete DBN model. The “static modulus”  $E(0)$  in equation (5) corresponds to the mix modulus when the pulsation  $\omega$  tends towards 0. It comes:

$$E(0) = \left( \sum_{i=0}^n \frac{1}{E_i} \right)^{-1} \quad (8)$$

Figure 8 presents simulated curves for the DBN with 15 elements and 2S2P1D models and experimental data, obtained for a 50/70 mix at 15°C. The constants ( $E_i$  and  $\eta_i$ ) for the DBN model are given in appendix A.



**Figure 8. Example of experimental and modeled complex modulus: master curves of the stiffness modulus  $|E^*|$  (up) and of the phase angle (down) at a reference temperature of 15°C.**

### *Calibration in the large strain domain and for a small number of cycles : non linearity and viscoplastic flow*

The compression and tension tests at constant strain rate and constant temperature on mixes show that the stress cannot exceed a flow stress  $\sigma_p$ . This ductile failure stress, observed only at intermediate and high temperature, is a function of strain rate ( $\dot{\epsilon}$ ) and of temperature (T). The viscoplastic criterion proposed by Di Benedetto (13) and Di Benedetto & Yan (16) was used. A tridimensional representation of this criterion is given hereafter. As for the considered one-dimensional case, the expression of the criterion is given by :



$$\sigma_p = \beta \ln \left( \frac{\dot{\varepsilon} + \delta}{\dot{\varepsilon}_0} \right) + \gamma \quad (9)$$

where  $\dot{\varepsilon}_0 = 1\% / \text{mn}$  ;  $\beta$ ,  $\delta$  and  $\gamma$  are three constants of the material.

When considering tensile tests at constant strain rate, the stress “ $\sigma_p$ ” is directly linked to the thresholds “ $s_i$ ” (see Figures 4.b and 5). The threshold in tension  $s_i^-$  are then calculated to obtain viscoplastic flow stress close to that given by the criterion (equation 9). First, the criterion constants are fitted from experimental results (tensile and/or compression tests can be considered). Then, the  $2n$  ( $s_i^+$ ,  $s_i^-$ ) values are obtained from the 3 constants of the viscoplastic criterion (equation 9) by an optimization process. It is considered that the threshold values in compression  $s_i^+$  are obtained from the values  $s_i^-$  by using the constant ratio parameter  $k$  already introduced in equations 2 and 3 ( $-k = s_i^+ / s_i^-$  for every  $i=1$  to  $n$ ) (Di Benedetto & Yan [16]).

### **Calibration for large strains at low temperatures : brittle failure**

At high temperatures, bituminous mixes have a purely ductile behavior (highly non linear stress-strain diagram with a viscoplastic flow), whereas at very low temperatures their behavior is purely brittle (linear stress-strain diagram with a catastrophic failure). At intermediate temperatures, their behavior slowly switches from ductile (high temperature) to brittle (low temperature), when decreasing the temperature (Figure 9).

Experimental results show that, as expected from the viscoplastic law formulation presented for instance in equation (9), the stress at mix failure is very dependent on the strain rate in the ductile domain at intermediate and high temperatures (Figure 10). Furthermore, Olard [4] and Olard et al. [17] [19] have previously established that, in a first approximation, the strain rate influence upon the determination of the mix tensile strength appears as negligible in the brittle domain, at low temperatures (Figure 10).

Under these circumstances, the brittle low-temperature failure can be modeled if it is introduced in the body EP0 (Figure 4.b), which then becomes elastic and brittle with a temperature-dependant failure limit. This behavior is represented Figure 9. The brittle stress at failure can be modeled by a function of the temperature with only two constants. Practically, at a given temperature  $T$ , the numerical simulation using the DBN law is stopped when the calculated stress  $\sigma$  exceeds the experimental tensile strength curve  $\sigma_c(T)$  at low temperatures (cf. Figure 11). A hyperbolic equation of the following type can be used :

$$\sigma_c(T) = \frac{a_1}{T + a_2} \quad (10)$$

The 2 constants of equation (10),  $a_1$  and  $a_2$ , can be obtained if the brittle stress is known at two different temperatures (e.g. at  $T_{\text{TSRST}}$  [TSRST fracture temperature] and  $T_{\text{TSRST}+5\text{C}}$ ).

Following the strain rate and the temperature, the stress level can reach or not the “brittle failure E0 limit”  $\sigma_c$ . If this limit is reached (for example at very low temperature), the failure is of the brittle type. If this limit cannot be reached because of viscoplastic flow (for example at high temperature and not too high strain rate), the failure is of the ductile type (noted  $\sigma_p$ ) (see Figure 9). Not only the temperature but also the strain rate level have a great influence upon the failure type. This modeling tendency corresponds to experimental observation as showed in the section presenting comparison between simulation and data.

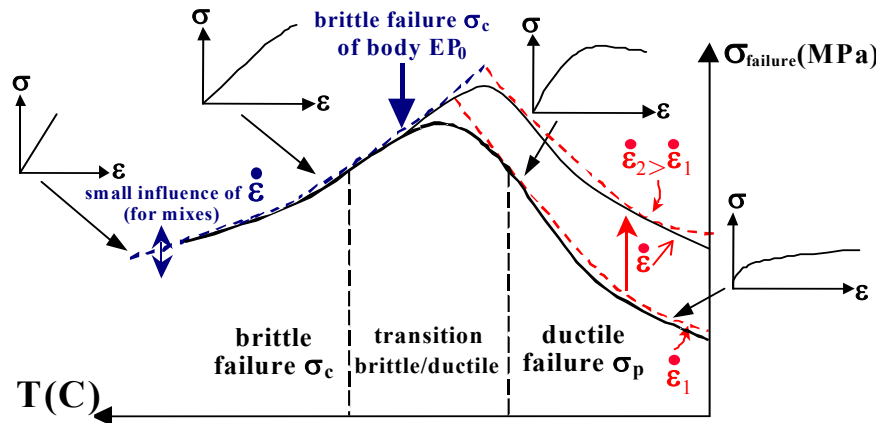


Figure 9. Schematic representation of the influence of temperature ( $T$ ) and strain rate ( $\epsilon$ ) on the failure behavior for tensile tests at constant strain rate on mixes.

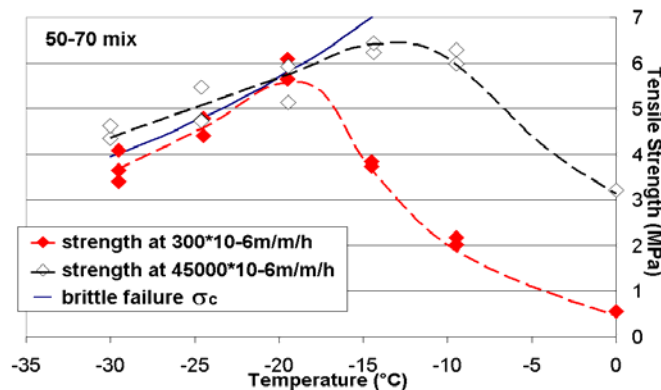


Figure 10. Effect of temperature and strain rate on the mix tensile strength for a given mix (Olard et al. [4] [17] [19] [27]).

### Remarks on the number of constants

From the calibration procedure presented in the three previous sections, it can be concluded that, whatever the number of elements “ $n$ ” chosen for the DBN model, the number of basic constants remains the same :

- 3 for the WLF equation (TTSP),
- 7 for the linear viscoelastic (LVE) domain,
- 3 for the viscoplastic flow,
- 2 for the brittle failure.

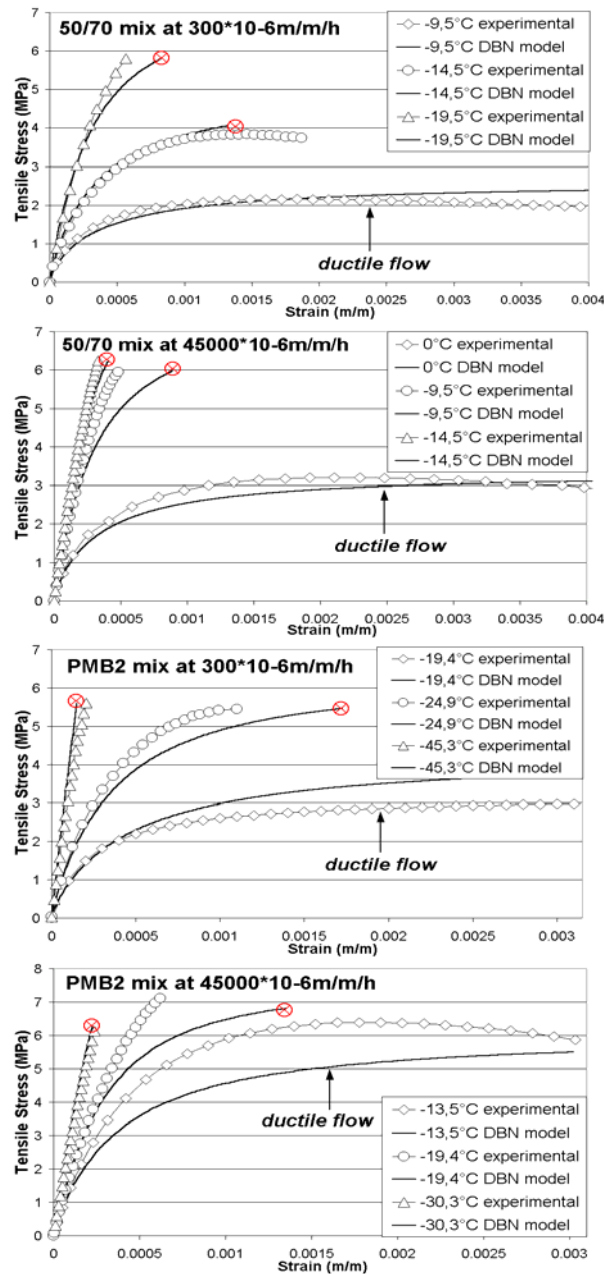
It is then up to the user to increase the number of elementary bodies for a more powerful description. As a counterpart, the calculation process will become longer.

### Experimental results Vs numerical simulations

To illustrate the DBN law efficiency for conditions other than those present during the model development stages, simulations of Direct Tensile Test (DTT) and of Thermal Stress Restrained Specimen Test (TSRST) are compared hereafter with the experimental data obtained on two mixes made from the same mix composition and two different binders called 50/70 and PMB2. The first bitumen is a 50/70 pen grade pure bitumen and the second is a highly polymer modified bitumen with a penetration of 88. The results are drawn from the PhD thesis of the first author [4]. Values of  $E_i$ ,  $\eta_i$ ,  $s_i^+$  and  $s_i^-$ , obtained from the calibration process on the two considered mixes, are given in Appendix A.

**DTT tests**

Figure 11 illustrates the comparison between the experimental DTT results and the corresponding predictions obtained from a discrete 15-element DBN model. In these simulations, the tensile stress is considered as positive so as to avoid the use of the negative sign and to conform to the hypothesis of the Continuous Media Mechanics. The obtained simulations are encouraging as they are very close to the experimental data. As previously evidenced by Di Benedetto et al. [14], Neifar et al. [10-11] and Olard [4], Figure 11 highlights that the chosen number of elements (n=15) is enough to describe the mix behavior accurately.



**Figure 11. Experimental results and numerical simulations using the DBN model for the two considered mixes (Olard et al. [4] [12]). Symbol ⊗ indicates the brittle failure obtained from the EP0 element.**

More generally, the DBN model allows to link the linear viscoelastic behavior for small strains and the viscoplastic flow of mixes. The formulation of this thermo-visco-elasto-plastic law remains simple as the two previous kinds of behavior are linked by hyperbolic relation. The model thus allows to describe a linear behavior in the small strain domain, a non linear behavior for higher strains and the viscoplastic flow. The temperature effect is described for small strains, where the Time-Temperature Superposition Principle (TTSP) is considered, and for higher strains as well (Figure 11). Only the viscosities  $\eta_i(T)$  and the brittle failure of the spring « E0 » are temperature dependant (Figure 4.b).

***Thermo-mechanical coupling : Thermal Stress Restrained Specimen Tests (TSRST)***

When considering modeling of restrained cooling tests (TSRST), the strain  $\varepsilon$  can be decomposed into the sum of a “thermal” strain ( $\varepsilon_{\text{thermal}}$ ) and a “mechanical” strain ( $\varepsilon_{\text{mechanical}}$ ) :

$$\varepsilon = \varepsilon^{\text{mechanical}} + \varepsilon^{\text{thermal}} \quad (11)$$

The thermal strain is equal to  $\alpha \cdot \Delta T$ , with  $\alpha$  the thermal contraction coefficient of the considered bituminous mixture. The mechanical part is given by the DBN model.

As the total strain  $\varepsilon$  is kept equal to zero during the whole test, equation (11) becomes :

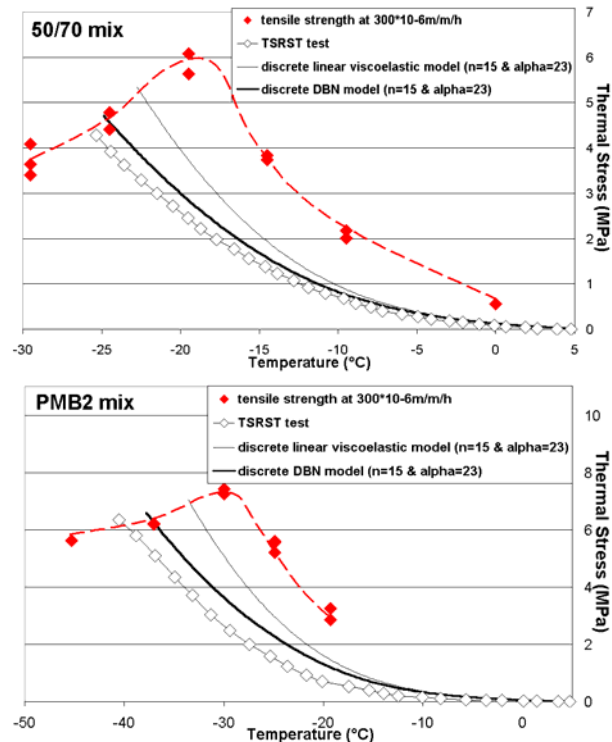
$$\varepsilon^{\text{mechanical}} = -\varepsilon^{\text{thermal}} = -\alpha \Delta T \quad (12)$$

In a complementary study, Olard et al. have measured the thermal contraction coefficient of the 50/70 and PMB2 mixes which varies from 30 to 15  $\mu\text{m}/\text{m}/\text{C}$  between +5 and -30C [4] [17]. Therefore, the equivalent mechanical strain rate ranges from 300 to 150  $\mu\text{m}/\text{m}/\text{h}$  during the restrained cooling tests for these two mixes.

As the material is restrained, its tendency to shorten results in the development of a thermal stress that produces failure.

*Simulation of TSRST with a monotonous temperature variation* Figure 12 presents a comparison between the experimental TSRST results obtained for the two considered mixes at a rate of -10C/h and the simulated stress evolution, when considering either a linear viscoelastic behavior (described by a generalized Kelvin-Voigt model with 15 elements) or a viscoplastic behavior (described by the DBN model with 15 elements).

It has to be emphasized again that, in order to introduce the brittle failure of mixes at low temperatures in the DBN model, it is considered that the EP0 element (Figure 4.b) has a brittle behavior. The brittle behavior failure value was calibrated from tension tests at constant temperature and strain rate.



**Figure 12. Experimental data and TSRST simulations at a rate of  $-10\text{C/h}$  when considering either a linear viscoelastic behavior or a viscoplastic one. The mean value of  $\alpha$  (over the studied temperature range) was around  $23\mu\text{m/m/C}$ .**

Figure 12 points out the influence of non-linearities to predict the stress evolution during TSRST correctly. The simulations which use a generalized Kelvin-Voigt model with 15 elements (linear viscoelastic behavior) lead to an overestimation (up to  $5\text{C}$ ) of the fracture temperature TTSRST.

During TSRST, the temperature field is not homogenous within the mix sample. Indeed, only the surface temperature is measured and the temperature in the center of the mix sample is slightly higher. A difference of around  $1.4\text{C}$  was measured by Olard [4] between the center and the surface of the sample. Then, if this correction of  $1.4\text{C}$  is applied to the results of Figure 12 (shift of the simulated curves of  $1.4\text{C}$ ), the comparison between the simulations and the experimental results shows that the DBN model gives a very good description of the thermal stress evolution during TSRST. These results appear as very encouraging.

The case of a cyclic temperature variation during the restrained cooling tests is treated in the following paragraph.

#### *Simulation of TSRST with a cyclic temperature variation*

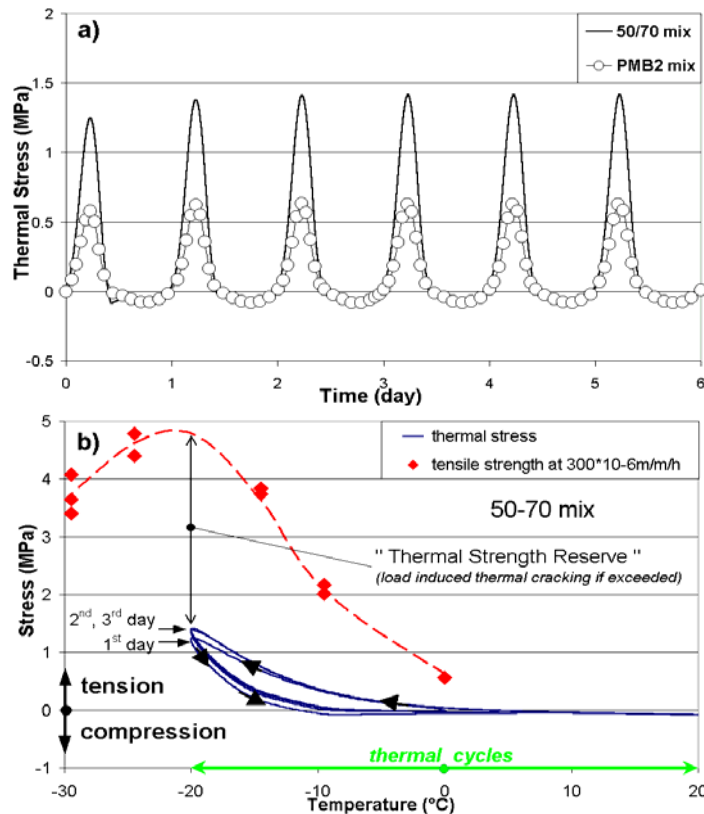
In this paragraph, a sinusoidal variation of the temperature is considered. It appears as interesting to quantify the effect of this type of variation on the thermally induced tensile stress during TSRST. The period of the cyclic variation is 24 hours with a view to approach the daily evolution on roads.

Figure 13.a shows the thermal stress evolution with time, for the 50-70 and PMB2 mixes, calculated thanks to the proposed DBN model. These simulations are carried out by considering a sinusoidal variation of the temperature between  $-20\text{C}$  and  $+20\text{C}$ .

It is noteworthy that the very strong non-symmetry between the tension phases (negative temperatures) and the compression phases (positive temperatures) is due to the great variation of the viscous properties according to the temperature.

The difference in stress levels between the two mixes is to underline. The PMB2 mix develops fewer stresses than the 50-70 mix which develops high tensile stresses as soon as the first cycles.

From Figure 13.a, one can finally observe that at the end of three to four thermal cycles (three to four days), the minimum and maximum stress levels are stabilized for each bituminous mix.



**Figure 13. a) Stress variation with time for two given mixes (example of a sinusoidal temperature variation between -20C and +20C, period = 1 day); b) Evolution and prediction of the mix thermal cracking during cyclic restrained cooling test.**

Figure 13.b illustrates the simulation of the thermal stress evolution with the temperature for the 50/70 mix. Besides, as the mix tensile strength (at 300 $\mu$ m/m/h) was obtained with a rather good repeatability, this figure also makes it possible to determine the difference between the tensile strength and the thermally induced tensile stress, i.e. the so-called thermal strength reserve. This figure highlights that there is hardly any risk of thermal cracking for this material.

Cyclic restrained cooling tests on these two mixes will be carried out to validate the efficiency of this simulation.

### Importance and bases for 3-Dimension generalization

The generalization to a tri-dimensional case of the previous developments is needed to model the road behavior correctly. This generalization needs the introduction of tensors instead of scalars for the different parameters. It implies that the experiment should not be analyzed in only one direction. Then, the volume variation and/or stress or strain evolution in

all directions have also to be measured. Di Benedetto et al. (1994) have previously developed a viscoplastic flow failure criterion, represented in Figure 14 [16].

An example of volume evolution during an haversine stress loading, for different stress amplitudes (tension and compression), temperatures and frequencies, is given in Figure 15 (Neifar et al. 2003 [28]). The contractancy and dilatancy domains for permanent deformation, can be observed. This kind of results are of primary importance for a correct description of the complex phenomena existing in the medium.

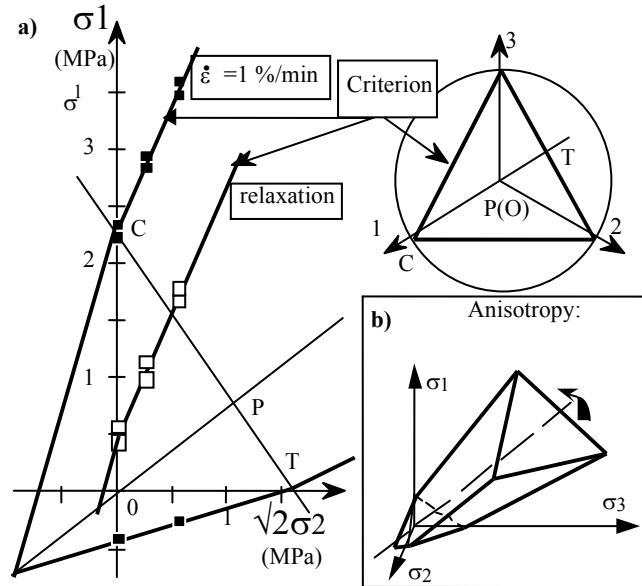


Figure 14. 3D failure criterion developed for viscoplastic flow : shape and data (Di Benedetto and Yan 1994 [16]).

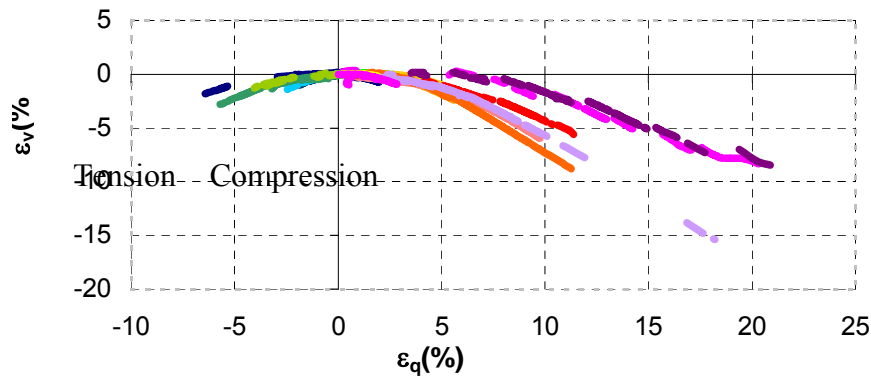


Figure 15. Volume variation vs. deviatoric ( $\epsilon_{axial-gradial}$ ) irreversible strain for haversine stress controlled tests at different maximum stresses, temperatures and frequencies (Neifar 2003 [28]).

### Future developments for the DBN model

Four other kinds of developments are being undertaken at the ENTPE for the DBN model so as to take into account respectively fatigue damage, accumulation of permanent deformation, thixotropy and healing of bituminous mixtures. The introduction of the appropriate improvement at the level of each  $E_{Pi}$  and  $V_i$  body of the DBN model is still part of our research work. Further details are given by Di Benedetto and Neifar in [29].

## Conclusion

Different aspects of the complex behavior of bituminous mixtures are presented in this paper. Typical types of mix behavior can be identified when considering the strain amplitude and the number of applied cycles (cf. Figure 3). A correct interpretation of the laboratory tests as well as a rational approach for the road design needs that the different facets of the behavior of the bituminous mixtures be modeled correctly.

This paper briefly presents the global law ("DBN law") developed in the laboratory. It is explained how the different kinds of behavior can be modeled using the same formalism but considering or not simplified formulation adapted to the required property. The law is very versatile and can be considered to model a wide range of characteristics such as :

- linear viscoelasticity,
- non linearity,
- viscoplastic flow,
- brittle failure,
- fatigue,
- accumulation of permanent deformation
- thixotropy,
- healing.

This general "DBN law" thus consists in a very effective alternative to the widespread procedures which are based only on the linear viscoelastic properties of the bituminous materials [30-33]. The DBN law allows to simulate the response for any imposed strain or stress path. In particular, some comparisons between experimental DTT and TSRST results and numerical simulations based on the DBN law are presented to illustrate the efficiency of the law.

Following the quality and the range of required description the expression of the law can be chosen as more or less complex. Asymptotic behaviors associated with simpler expressions can be obtained in a continuous way when the loading conditions give a "typical" type of behavior.

Three-dimension generalization of the law is introduced. Experimental results integrating the whole directions are of main importance and needed to allow this generalization.

The model formalism allows to consider any number of elementary bodies, without changing the number of basic constants. It is then easy to choose a more powerful description by increasing the number of elementary bodies.

Some of the presented expressions show the general principle and need more additional research developments. Some additional details on fatigue damage, accumulation of permanent deformation, thixotropy and healing are, for instance, given by Di Benedetto and Neifar in [29].

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### Appendix A

Values of the parameters  $E_i$ ,  $\eta_i$  (at T=15C),  $s_i^+$  and  $s_i^-$  of a 15-element DBN model for the 50/70 mixture.

Element number	$E_i$ (MPa)	$\eta_i$ (MPa*s)	$s_i^+$ (MPa)	$s_i^-$ (MPa)
<b>0</b>	<b>45000</b>	—		
<b>1</b>	<b>381</b>	80000	2.86	-0.80
<b>2</b>	1200	20000	5.72	-1.60
<b>3</b>	3700	10000	5.72	-1.60
<b>4</b>	10500	8000	5.72	-1.60
<b>5</b>	13500	4000	5.72	-1.60
<b>6</b>	17300	500	8.23	-2.30
<b>7</b>	47400	100	15.42	-4.32
<b>8</b>	130000	30	20.00	-5.60
<b>9</b>	160000	9	24.44	-6.84
<b>10</b>	175000	0.6	28.59	-8.00
<b>11</b>	375210	0.05	39.28	-11.00
<b>12</b>	640000	0.005	46.97	-13.15
<b>13</b>	1300000	0.0005	55.17	-15.45
<b>14</b>	1830000	0.00005	63.05	-17.65
<b>15</b>	2960000	<b>2.90E-06</b>	71.07	-19.90

Values of the parameters  $E_i$ ,  $\eta_i$  (at T=15C),  $s_i^+$  and  $s_i^-$  of a 15-element DBN model for the PMB2 mixture.

Element number	$E_i$ (MPa)	$\eta_i$ (MPa*s)	$s_i^+$ (MPa)	$s_i^-$ (MPa)
<b>0</b>	<b>44000</b>	—		
<b>1</b>	<b>657</b>	77983	3.07	<b>-0.86</b>
<b>2</b>	700	6880	6.13	<b>-1.72</b>
<b>3</b>	1918	2293	6.13	<b>-1.72</b>
<b>4</b>	5442	1835	6.13	<b>-1.72</b>
<b>5</b>	6997	917	6.13	<b>-1.72</b>
<b>6</b>	8966	115	6.13	<b>-1.72</b>
<b>7</b>	24567	23	12.13	-3.40
<b>8</b>	67377	6.88	15.89	-4.45
<b>9</b>	82926	2.06	19.55	-5.47
<b>10</b>	90700	0.14	22.95	-6.43
<b>11</b>	194467	0.011	31.76	-8.89
<b>12</b>	331704	0.0011	38.09	-10.66
<b>13</b>	673774	0.00011	44.83	-12.55
<b>14</b>	948466	1.14E-05	51.31	-14.37
<b>15</b>	1534131	6.66E-07	57.91	-16.22

## Annexe 2

### Influence des propriétés élastiques ou viscoélastiques des revêtements sur le comportement des ponts à dalle orthotrope (2007-2011)

Deux articles de rang A sont ici fournis pour illustrer l'originalité du travail:

- S. Pouget, C. Sauzéat, H. Di Benedetto, F. Olard, Numerical simulation of the five-point bending test designed to study bituminous wearing courses on orthotropic steel bridge, *Materials and structures*, Vol. 43, n°3, pp. 319-330, 2010.
- S. Pouget, C. Sauzéat, H. Di Benedetto, F. Olard, Effect of Vehicle Speed on the Millau Viaduct Response, *Journal of Testing and Evaluation (JOTE)*, Vol. 40, N° 7, 2012.



Materials and Structures (2010) 43:319–330  
DOI 10.1617/s11527-009-9491-1

ORIGINAL ARTICLE

## Numerical simulation of the five-point bending test designed to study bituminous wearing courses on orthotropic steel bridge

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**Abstract** The structural role of the wearing courses on orthotropic steel deck bridges is usually not estimated during the design process. Both geometry of the structure and very high flexibility of steel plates induce severe stress and strain fields in the steel bridge surfacing. In this paper, we quantify the influence of a bituminous mix surfacing on the steel plate response when considering the French Five-Point Bending Fatigue Test. Mechanical behavior of

bituminous mixes is very complex. It includes a great thermal sensitivity. Moreover in the small strain domain (for strain amplitudes below about  $10^{-4}$ ) the behavior can be considered as linear. Therefore, the theory of linear viscoelasticity can be applied. After introducing the French five-point bending fatigue test used at the research center of “EIFFAGE Travaux Publics” company, in parallel to the construction of the Millau Viaduct (France) –the highest bridge in the world–, a numerical parametric study using the Finite Element Method (FEM) is presented. The specimens are made of a 12-mm-thick steel plate, a 3-mm-thick sealing sheet and a 60-mm-thick bituminous mix. Different wearing course behaviors are considered introducing temperature and viscous effects. This behavior is modeled using the linear part of the general thermo-viscoplastic model «DBN» (Di Benedetto and Neifar), recently developed at ENTPE laboratory and briefly described in this paper. An important observation is that the rigidity of the intermediate bituminous sealing sheet as well as the wearing course behavior seems to have a great influence on the orthotropic multilayer specimen complex behavior.

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**Keywords** Orthotropic steel bridges deck ·  
Bituminous mix · French Five-Point Bending  
Test · Viscoelasticity · FEM ·  
Bituminous sealing sheet



## 1 Introduction

### 1.1 Context

When the weight of a bridge is a determining parameter for its optimization (long-span bridges), an orthotropic steel deck bridge often constitutes the best adapted solution [1]. If the general response of orthotropic structures is relatively simple to understand (type “frame of beams”), the fine modeling of their behavior is very complex. Note that “orthotropic” is a combination of two words, orthogonal and isotropic, and is a contraction of the expression “orthogonally anisotropic.” Yet their important deformability makes them particularly sensitive to fatigue phenomena. Different pathologies connected to fatigue were noticed in the past on this kind of structure [2]. Because of the already evoked difficulties of modeling, these pathologies were avoided by the adoption of safe well-known constructive rules. Such an approach leads naturally to a design is not optimized although satisfactory [3, 4].

For years, observation was made that the wearing course seems to play an important part in the behavior of orthotropic steel deck bridges (distribution of the concentrated loads, and participation in local bending) [5]. Despite that, constraining design practices were adopted. The recent progress in calculation methods make possible a precise analysis of the beneficial effect of the surfacing on orthotropic plate, which is desirable considering the weight requirements often imposed on this kind of structure. It really becomes an element of the structure. Design of orthotropic deck could then include not only the geometry of spans and girders and the thickness of the deck plate, but also the properties of the surfacing material as well as its thickness.

### 1.2 Objectives

The objectives of our research, which is partly presented in this paper, are to use and develop theoretical and methodological tools allowing a realistic calculation of the life cycle of orthotropic steel deck bridges, taking into account the surfacing layer. The developed tools will also be used to optimize the wearing courses composed of bituminous mixes (nature, thickness), which have a very complex mechanical behavior. One of them used by

the EIFFAGE Travaux Publics laboratory for the Millau Viaduct design is an original test, called “French five-point bending test” (French standard NF P 98-286) [6–10]. It allows the choice of the pavement system for this kind of structure [5].

This paper presents some recent numerical investigations from Finite Element Method (FEM) calculations on the French five-point bending test performed at “ENTPE” (French acronym for “National School of Public Works”) Laboratory. After reviewing the main degradation modes for orthotropic steel plate surfacing and introducing the test principle, a parametric study is presented, which aims at checking the influence on the tested sample response of the surfacing behavior.

This study is included in a French national project called “Orthoplus”: Advanced engineering of orthotropic decks and their wearing courses for a global optimization of their life-cycle.

## 2 Review of the main degradation modes for orthotropic steel bridge

### 2.1 Deck

Both geometry of the structure and low thickness of the steel plates (between 10 and 16 mm) induce very high flexibility of orthotropic bridge deck. Under traffic loads fatigue phenomenon appears. The cracks which can appear in the deck are generally divided into four categories [2]:

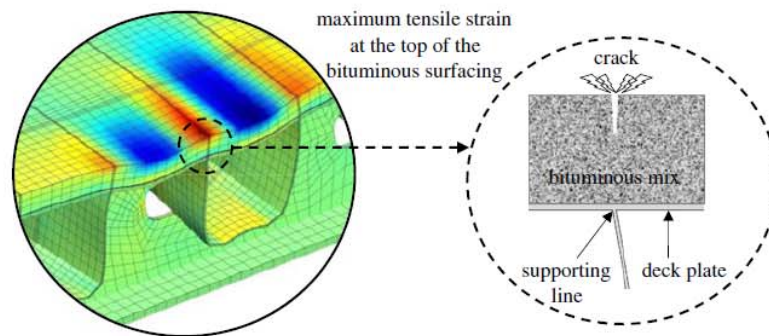
- Cracks in the deck plate;
- Cracks in the longitudinal weld between deck plate and trough web;
- Cracks in the trough splice joint;
- Cracks in the connection between trough profile and crossbeam.

### 2.2 Surfacing

During passing of live loads on orthotropic bridge, tensile stresses are generated in the upper part of the plate as well as on the top of the bituminous wearing course (Fig. 1). The surfacing is then subjected to fatigue damage due to repeated tension cycles. Nevertheless, unlike with traditional pavements, tension occurs in its uppermost part, bringing a risk



Fig. 1 Crack observed on bituminous mix surfacing of orthotropic bridge deck (adapted from [1])



of crack at the top of the wearing course. The extension strains measured at the top of the wearing course can exceed  $500 \mu\text{m}/\text{m}$ , which is definitely higher than those observed at the bottom of conventional bituminous pavements. This explains why most problems reported in the literature for orthotropic steel bridges, are related to fatigue cracking of bituminous mixes [1–5].

### 3 The French five-point bending test [10]

#### 3.1 Test principle [5, 8–14]

The French five-point bending test, developed by “Laboratoire Central des Ponts et Chaussées” (LCPC) in the 70s, allows the assessment of the fatigue resistance of steel bridge pavement under negative moment (over stiffeners) [6]. The simulation test consists in putting a pavement system composed of a steel plate, a sealing sheet, and a bituminous surfacing on three supports (including one embedded in the center). Two equal cyclic vertical loads are applied on the top of the sample by using two shoes fitted with rubber (Fig. 2 and Table 1).

#### 3.2 Calibration phase: determination of $F_{\text{max}}$

Testing procedure begins with a calibration phase in order to determine the  $F_{\text{max}}$  load. The steel plate is loaded to obtain a strain of  $625 \mu\text{m}/\text{m}$  on the surface over the central web welds, for a 12-mm-thick steel plate [10]. The corresponding force is  $F_{\text{max}} = 37.5 \text{ kN}$  which gives  $18.75 \text{ kN}$  on each shoe (corresponding to a pressure of  $0.72 \text{ MPa}$ ) [8]. This last

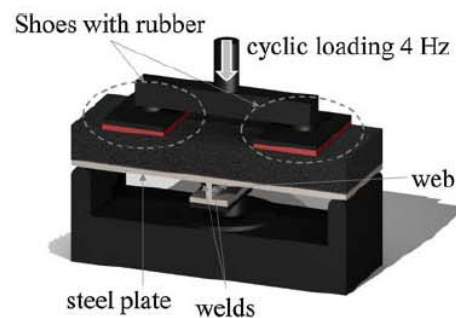


Fig. 2 Five point bending test: above, schematic; below, picture (dimensions are given in Table 1) (adapted from [7])

value is close to the French standard truck axle pressure,  $0.67 \text{ MPa}$ .

#### 3.3 Observations during the test

During the five-point bending test, damage is quantified by means of rupture witnesses and also trough strain gauges placed in the central zone on upper and lateral sides of the mix. Observations on the upper



**Table 1** Dimensions and testing parameters

Sample dimensions	580 × 200 mm
Supports location	270 mm between extreme and central supports
Shoes dimensions	130 × 200 mm
Shoes location	65 mm from the ends of the samples
Sinusoidal compression load	Ranging between $F_{\max}$ and $0.1 \times F_{\max}$
Frequency	4 Hz
Temperatures	+10 and $-10^{\circ}\text{C}$
Number of cycles	2 million at $+10^{\circ}\text{C}$ 1 million at $-10^{\circ}\text{C}$

side include detection of the appearance of a crack in the central zone and follow-up of its propagation toward the edges of the sample. Observations on the lateral sides include detection of delamination at the steel-sealing and sealing-mix interfaces and crack depth measurement in the bituminous mix.

#### 3.4 Acceptance criteria

Regarding the case of the Millau Viaduct, technical requirements were to obtain at the end of the test: (i) no separation between steel/sealing and sealing/mix interfaces, (ii) least crack depth in the mix at  $10^{\circ}\text{C}$  and 2 million cycles, and at  $-10^{\circ}\text{C}$  and 1 million cycles.

## 4 Numerical simulations

Finite Element simulations are performed using “COMSOL” software considering 2D plane strain hypothesis. This study allows the evaluation of the

stress and the strain levels in the specimen during the test. Different wearing course behaviors (Table 3) are proposed introducing temperature and viscous properties. In the next section, the tension is considered as positive. Moreover, perfect bond is assumed between layers.

### 4.1 Hypotheses for the numerical simulations

#### 4.1.1 Geometry

The specimen (Fig. 3) is made of:

- a 12-mm-thick steel plate, which is a classical thickness used for fast lane on orthotropic steel bridge with a bituminous surfacing;
- a 3-mm-thick sealing sheet which provides an impermeable barrier and absorbs differential movements between the surface course and the steel plate;
- a 60-mm-thick bituminous mix.

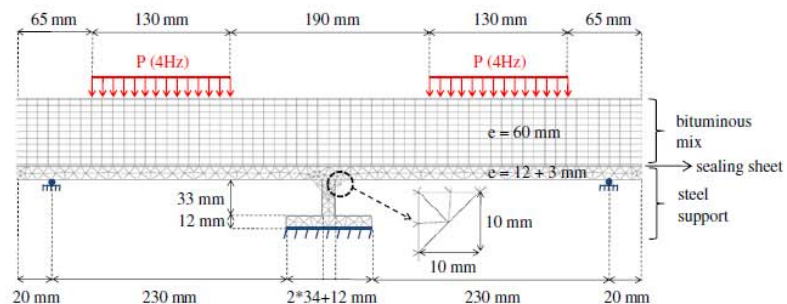
#### 4.1.2 Boundary conditions (Fig. 3)

The bottom central support is considered as clumped while the two others allow free horizontal movement. The sinusoidal compression loading  $P$  is ranging between 0 and  $-0.72\text{ MPa}$  at a frequency of 4 Hz. The simulation is made for 10,000 s which corresponds to 40,000 cycles (Fig. 3).

#### 4.1.3 Meshing

The mesh has 1,014 elements which correspond to 7,150 degrees of freedom (resp. 28,430) when considering a linear elastic behavior (resp. a Linear viscoelastic behavior) for the bituminous mix

**Fig. 3** Geometry, boundary conditions and FEM mesh of the specimen



surfacing. The mesh of the surfacing and of the sealing sheet is constituted by quadrangle elements while the mesh of the steel plate is composed of triangle elements.

#### 4.2 Wearing course: bituminous mix behavior

Mechanical behavior of bituminous mixes is very complex. It includes a great thermal sensitivity. In order to characterize this structural effect and its evolution with time, mechanical properties of bituminous mixes have to be modeled considering the following aspects:

- stiffness and stiffness evolution with time in the linear domain;
- fatigue and damage law evolution;
- permanent deformation;
- cracks and crack propagation, in particular at low temperature.

These four properties are of main interest when modeling the pavement behavior. The first one is observed for very small strain amplitudes (less than  $10^{-4}$  m/m) and corresponds to the linear viscoelastic (LVE) behavior of mixes. The three other aspects are at the origin of major distresses [15–18]. Each of these properties or distresses appears for a given domain of loading and corresponds to a specific type of behavior for the mixes. Temperature is a parameter of paramount importance in the description of these types of bituminous mixes behavior (Fig. 4).

In this paper only the linear viscoelastic (LVE) behavior is considered. No non-linearity (fatigue, permanent deformations, cracks) was taking into account.

The considered M1B9 bituminous material is an asphalt mix with 6% (aggregate weight) of 50/70

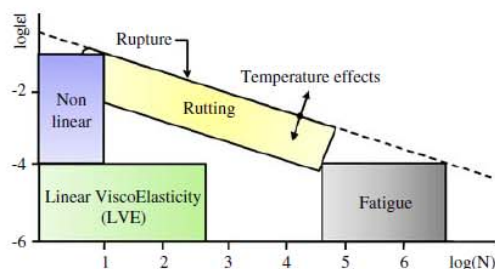


Fig. 4 “Typical” behavior observed on bituminous mixes ( $\epsilon$ ) strain—( $N$ ) number of loadings [19]

penetration grade bitumen and a continuous 0/10 mm aggregates grading. Its behavior in the small strain domain (LVE) is completely defined by the complex Young’s modulus  $E^*$  and by the complex Poisson’s ratio  $\nu^*$  master curves plotted at a reference temperature ( $T_{ref}$ ) of  $10^\circ\text{C}$  (Fig. 5). The hypothesis of isotropy is postulated.

It can be seen that the norm of Poisson’s ratio  $|\nu^*|$  is not a constant equal to 0.35 as generally admitted in the literature.  $|\nu^*|$  varies between about 0.25 for high frequencies and/or low temperatures and 0.5 for low frequencies and/or high temperatures. It decreases when temperature decreases and when frequency increases.

The Time Temperature Superposition Principle (TTSP) is generalized in the 3D case as shown by

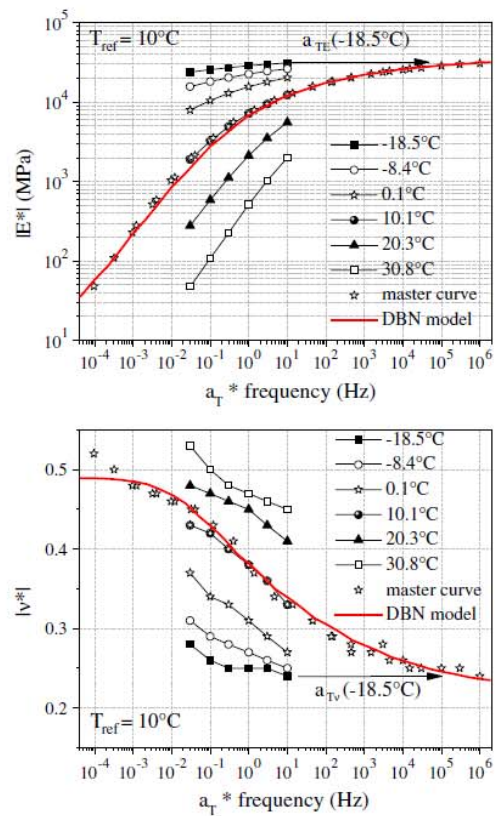
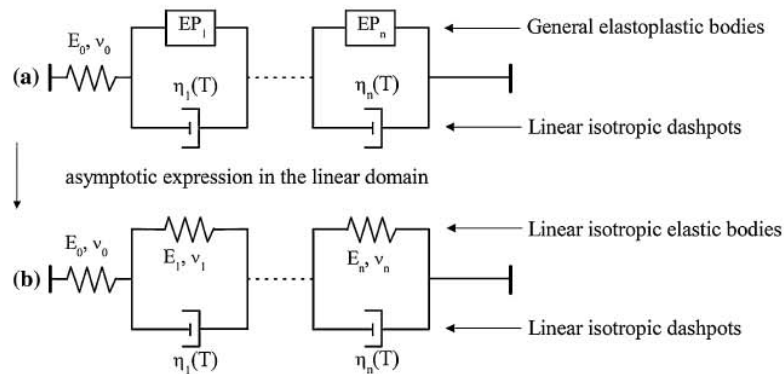


Fig. 5 Experimental master curves (test M1B9) and DBN model simulation (20 elements) (Fig. 6) plotted at a reference temperature of  $10^\circ\text{C}$  [20]. Above: norm of complex Young’s modulus  $E^*$ ; below: norm of complex Poisson’s ratio  $\nu^*$



Fig. 6 a DBN model for bituminous mixes; b asymptotic expression in the linear domain (equivalent to generalized Kelvin-Voigt model)



[20–22]. A unique shift factor is considered for  $E^*$  and  $\nu^*$  ( $a_T = a_{T\nu} = a_{TE}$ ).

The classical WLF law (William, Landel and Ferry) [23] is used to fit the shift factor  $a_T$ :

$$\log(a_T) = -\frac{C_1(T - T_{ref})}{C_2 + T - T_{ref}}$$

at  $T_{ref} = 10^\circ\text{C}$  :  $C_1 = 25$  and  $C_2 = 170$

(1)

The DBN (Di Benedetto–Neifar) model has been developed at the ENTPE (Ecole Nationale des

Travaux Publics de l’Etat) during the last decades to simulate the behavior of bituminous materials. This model is an attempt to describe with a unique formulation the complex elasto-visco-plastic behavior observed for bituminous materials. Due to the lack of place only a rapid introduction of the model is given in this paper. The reader could refer to [16–18, 24], for more details.

The general analogical form of the DBN model is presented Fig. 6. It consists in an assembly in series of a linear isotropic elastic body and  $n$  elements

Table 2 Parameters of the DBN model (Fig. 6) for the bituminous mix used for the numerical simulations at two temperatures: 10 and 30°C

Body n°	$E_i$ (MPa)	$\nu_i$	$\eta_i$ at 10°C (MPa s)	$\eta_i$ at 30°C (MPa s)
0	3.60E+04	0.220		
1	1.45E+06	0.496	3.59E−05	8.62E−02
2	1.37E+06	0.383	3.72E−04	8.93E−01
3	1.07E+06	0.428	3.20E−03	7.68E−00
4	6.92E+05	0.439	2.27E−02	5.45E+01
5	4.25E+05	0.442	1.53E−01	3.67E+02
6	2.60E+05	0.448	1.03E+00	2.47E+03
7	1.56E+05	0.455	6.80E+00	1.63E+04
8	8.88E+04	0.463	4.27E+01	1.02E+05
9	4.58E+04	0.471	2.42E+02	5.81E+05
10	1.98E+04	0.479	1.15E+03	2.76E+06
11	6.87E+03	0.489	4.39E+03	1.05E+07
12	1.96E+03	0.495	1.38E+04	3.31E+07
13	4.85E+02	0.494	3.75E+04	9.00E+07
14	9.69E+01	0.491	8.25E+04	1.98E+08
15	2.11E+01	0.490	1.98E+05	4.75E+08
16	2.12E+02	0.490	2.18E+07	5.23E+10
17	7.19E+02	0.490	8.15E+08	1.96E+12
18	1.89E+04	0.490	2.35E+11	5.64E+14
19	4.95E+05	0.490	6.78E+13	1.63E+17



**Table 3** Description of the different bituminous mix behaviors used for the numerical calculations

Name of the calculation	Bituminous mix behavior	Considered temperature (°C)	$E$ (MPa)	$\nu$
LE1	Linear elastic	10	$E = 10,000$ $= E^*(10^\circ\text{C}, 4 \text{ Hz})$	$\nu = 0.35$ $= \nu^*(10^\circ\text{C}, 4 \text{ Hz})$
LE2	Linear elastic	30	$E = 1,000$ $= E^*(30^\circ\text{C}, 4 \text{ Hz})$	$\nu = 0.47$ $= \nu^*(30^\circ\text{C}, 4 \text{ Hz})$
LE3	Linear elastic	30	$E = 1,000$ $= E^*(30^\circ\text{C}, 4 \text{ Hz})$	$\nu = 0.35$ $= \nu^*(10^\circ\text{C}, 4 \text{ Hz})$
LVE1	Linear viscoelastic with time-dependent $\nu$	10	From Fig. 5 (DBN model)	From Fig. 5 (DBN model)
LVE2	Linear viscoelastic with time-dependent $\nu$	30	From Fig. 5 (DBN model)	From Fig. 5 (DBN model)
LVE3	Linear viscoelastic with constant $\nu$	10	From Fig. 5 (DBN model)	$\nu = 0.35$ $= \nu^*(10^\circ\text{C}, 4 \text{ Hz})$
LVE4	Linear viscoelastic with constant $\nu$	30	From Fig. 5 (DBN model)	$\nu = 0.35$ $= \nu^*(10^\circ\text{C}, 4 \text{ Hz})$

composed of one EP type body in parallel with one linear isotropic dashpot. It should be underlined that the number of elements  $n$  is a free choice from the user.

The EP<sub>*i*</sub> type bodies have a non-viscous behavior (independent of the temperature and of the loading rate). Their formulation can be adapted following the required properties to be introduced (viscoelasticity, non-linearity, viscoplasticity...).

The linear isotropic dashpots are only characterized with a viscosity ( $\eta_i$ ), which depends on the temperature and they respect the Time-Temperature Superposition Principle. All the dashpots have the same shift factor  $a_T$  ( $a_{T_i} = a_T$ ). The viscous effects and the thermal sensitivity are only introduced by these dashpots.

If small amplitude loadings are applied, the observed behavior is linear and is modeled by the DBN asymptotic form, presented Fig. 6b. It consists of a generalized Kelvin-Voigt model of  $n$  elements having each a Young's modulus  $E_i$ , a Poisson's ratio  $\nu_i$  and a viscosity  $\eta_i$ . The calibration of the constants ( $E_i$ ,  $\nu_i$ ,  $\eta_i$ ) is made from an optimization procedure in the frequency domain. The chosen constants at a reference temperature of 10°C are given in Table 2 for the considered mix (material M1B9 Fig. 5). Simulation with DBN model is also presented Fig. 5 for material M1B9.

The calculations are performed at 10 and 30°C in order to underline the influence of temperature and the viscous effects. The linear expression of the DBN model (Fig. 6b) is used in the Finite Element simulation. The considered constants at the two

temperatures are given in Table 2 for the bituminous wearing course (material M1B9 Fig. 5).

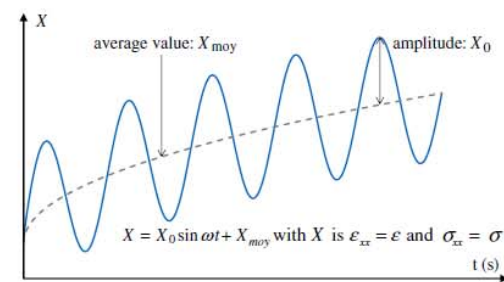
Calculations considering linear elastic properties for bituminous surfacing are also proposed to compare both hypotheses: linear elastic (LE) and linear viscoelastic (LVE). The elastic parameters are obtained from the master curves (Fig. 5) at the fixed considered temperature (10 or 30°C) for a frequency of 4 Hz.

The different characteristics of the bituminous surfacing considered for the simulations are indicated in Table 3.

#### 4.3 Steel and sealing sheet behavior

Steel and bituminous sealing sheet are assumed to be isotropic linear elastic materials.

The steel elastic constants ( $E$  and  $\nu$ ) are  $E = 210$  GPa and  $\nu = 0.3$ . To check the influence of the



**Fig. 7** Schematic evolution of strain ( $\epsilon$ ) and stress ( $\sigma$ ) with time



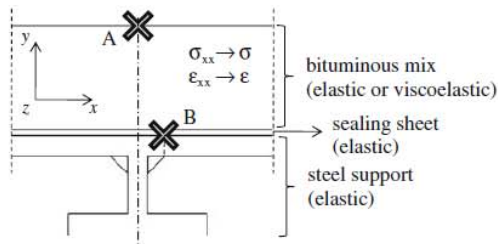


Fig. 8 Central part of the tested specimen (Fig. 3): location of point A in bituminous surfacing and point B in steel support where the FEM results are analyzed

sealing sheet stiffness ( $E_{ss}$ ), different values are successively used for calculations:  $E_{ss} = 1, 10, 100$  and  $1000$  MPa. The Poisson's ratio is fixed at  $0.45$  ( $\nu_{ss} = 0.45$ ).

#### 4.4 Numerical parametric study

The aim of the parametric study is to check the influence of the bituminous surfacing behavior as well as the influence of the sealing sheet stiffness on the structure response. Comparisons of the calculated normal strain in the horizontal direction ( $\epsilon_{xx}$  noted  $\epsilon$ ) and normal stress in the horizontal direction ( $\sigma_{xx}$  noted  $\sigma$ ) values at point B of the steel plate and at point A on top of the bituminous mix are considered (Fig. 8). These two points are chosen because the tension is maximum in bituminous mix (point A) and steel (point B). They correspond to the main critical points for this type of bridge structure [5, 8].

Schematic evolution of strain and stress are given Fig. 7. The amplitude (noted  $\epsilon_0$  and  $\sigma_0$ ) as well as the average values (noted  $\epsilon_{moy}$  and  $\sigma_{moy}$ ) of the materials' responses ( $\epsilon$  and  $\sigma$ ) at points A and B (Fig. 8) are presented at cycle 40,000 in Figs. 9, 10, 11, 12.

##### 4.4.1 Analysis of steel response at point B for cycle 40,000 as a function of sealing sheet stiffness

Due to the Linear Elastic (LE) steel behavior hypothesis, the values of stress ( $\sigma$ ) and strain ( $\epsilon$ ) are proportional and the ratio is equal to the chosen Young's Modulus (210 GPa). The effect of stress in directions  $y$  and  $z$  can be neglected.

Amplitude  $X_0$  (Fig. 9 above and Fig. 10 above). It can be noticed that the amplitudes ( $\epsilon_0$ ;  $\sigma_0$ ) are higher at

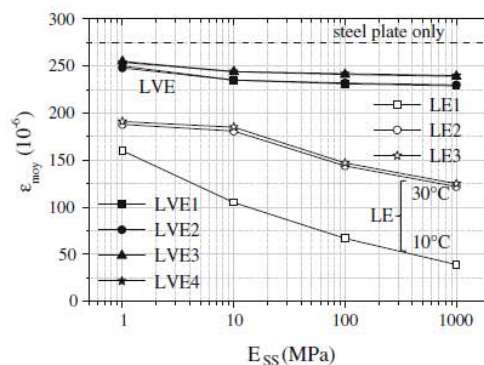
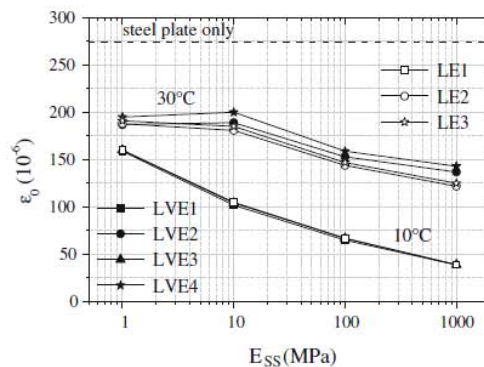


Fig. 9 Strain values at 10,000 s (40,000 cycles) in the steel plate at point B (Fig. 8) versus sealing sheet stiffness ( $E_{ss}$ ) considering different temperatures (10 and 30°C) and different wearing course behaviors (Table 3). Above: amplitude of  $\epsilon$ ; below: average values of  $\epsilon$

30°C than at 10°C. But both are smaller than results obtained without surfacing. The calculations reveal very little influence of the Poisson's ratio at 10°C. At 30°C, both influence of the chosen constant value as well as the effect of time-dependent Poisson's ratio are observed.

The variation of the sealing sheet stiffness ( $E_{ss}$ ) does not give the same effects at 10 and 30°C. Indeed, at 10°C, the amplitude  $X_0$  still decreases with the increase of  $E_{ss}$ . On the other hand, at 30°C the amplitudes reach a maximum value at about  $E_{ss} = 10$  MPa.

Average value  $X_{moy}$  (Fig. 9 below and Fig. 10 below). The average values ( $\epsilon_{moy}$ ,  $\sigma_{moy}$ ) obtained in the elastic cases (LE1, LE2 and LE3) are higher at 30°C than at 10°C. This is not true in the LVE cases (LVE1, LVE2, LVE3 and LVE4). Due to the relaxation of the bituminous surfacing, the results at 30°C and at 10°C are close. But both are smaller than results obtained



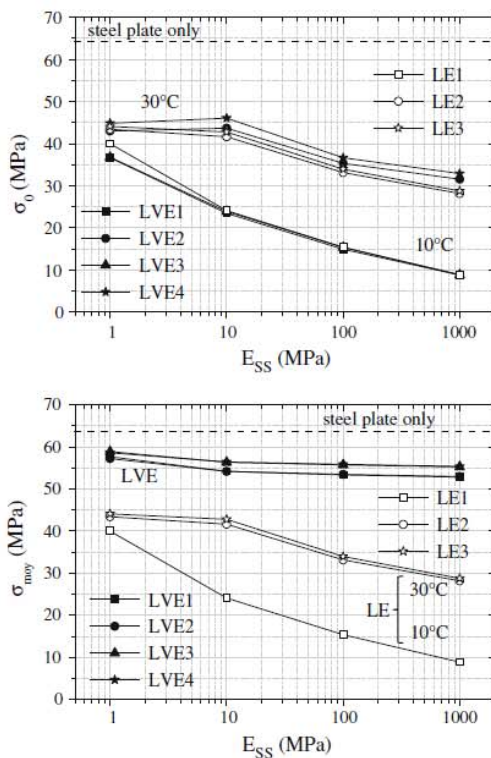


Fig. 10 Stress values at 10,000 s (40,000 cycles) in the steel plate at point B (Fig. 8) versus sealing sheet stiffness ( $E_{SS}$ ) considering different temperatures (10 and 30°C) and different wearing course behaviors (Table 3). Above: amplitude of  $\sigma$ ; below: average values of  $\sigma$

without surfacing indicated by a dashed line in the figures. The calculations reveal very little influence of the Poisson's ratio in the LE case. In the LVE case, differences appear between the results obtained with the constant Poisson's ratio hypothesis and the time-dependent Poisson's ratio hypothesis.

In the LVE case the average values still decrease with the increase of  $E_{SS}$  but seems to reach a constant value independent of the sealing sheet stiffness.

#### 4.4.2 Analysis of bituminous mix response at point A for cycle 40,000 as a function of sealing sheet stiffness

Due to the Linear ViscoElastic (LVE) bituminous mix behavior hypothesis, the values of stress ( $\sigma$ ) and strain ( $\varepsilon$ ) are not proportional anymore.

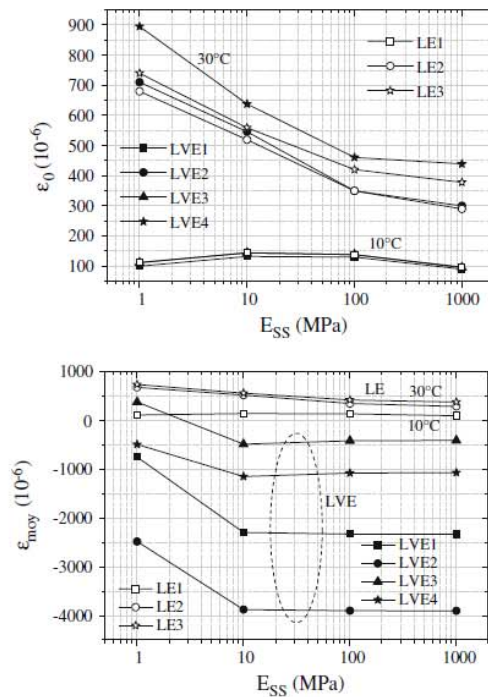


Fig. 11 Strain values at 10,000 s (40,000 cycles) in the bituminous wearing course at point A (Fig. 8) versus sealing sheet stiffness ( $E_{SS}$ ) considering different temperatures (10 and 30°C) and different wearing course behaviors (Table 3). Above: amplitude of  $\varepsilon$ ; below: average values of  $\varepsilon$

#### Amplitude $X_0$ (Fig. 11 above and Fig. 12 above).

It can be noticed that the strain amplitudes ( $\varepsilon_0$ ) are higher at 30°C than at 10°C. However the stress amplitudes ( $\sigma_0$ ) are higher at 10°C than at 30°C. As previously, in the steel at point B, the calculations reveal minimal influence of the Poisson's ratio at 10°C. At 30°C, both an influence of the chosen constant value as well as the effect of time-dependent Poisson's ratio are observed.

Contrary to the results in the steel at point B, the computations at point A exhibit a constant decrease of the amplitude values  $X_0$  at 30°C with the increase of  $E_{SS}$ . At 10°C, the amplitudes reach a maximum value at about  $E_{SS} = 10$  MPa.

Average value  $X_{moy}$  (Fig. 11 below and Fig. 12 below). The average strain values ( $\varepsilon_{moy}$ ) obtained in the elastic cases (LE1, LE2 and LE3) are higher at 30°C than at 10°C. However the average stress values ( $\sigma_{moy}$ ) are higher at 10°C than at 30°C.  $\varepsilon_{moy}$  is

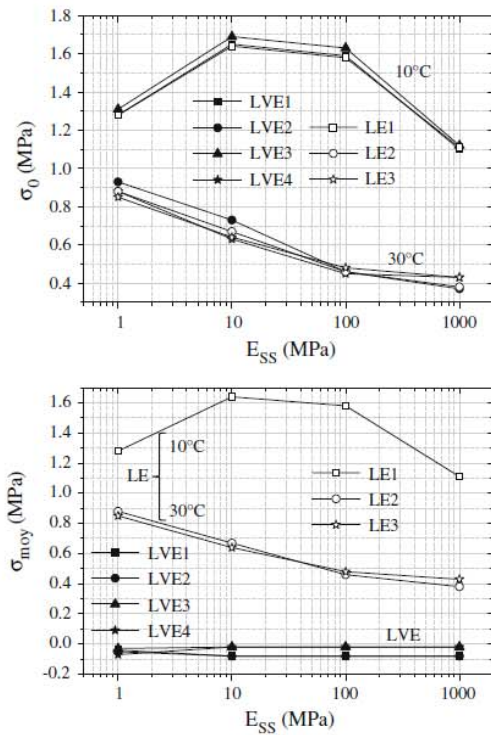


Fig. 12 Stress values at 10,000 s (40,000 cycles) in the bituminous wearing course at point A (Fig. 8) versus sealing sheet stiffness ( $E_{SS}$ ) considering different temperatures (10 and 30°C) and different wearing course behaviors (Table 3). Above: amplitude of  $\sigma$ ; below: average values of  $\sigma$

different for each of the four LVE calculations and is negative (contraction), which is not the case for LE behavior, where the strain is always positive (extension). This observation is of utmost importance and is discussed more in details in the next section. The average stress values of LVE calculations are also negative (compression) at 30°C and at 10°C. Their values are comparable and close to zero. In the LE case at 30°C and in LVE case at the two temperatures (10 and 30°C), differences appear between the results obtained with constant Poisson’s ratio hypothesis and time-dependent Poisson’s ratio hypothesis. This underlines the important effect of chosen hypothesis for Poisson’s ratio.

In the LVE case the average values still decrease with the increase of  $E_{ss}$  and rapidly tend to become stable.



#### 4.4.3 Differences between LVE behavior and LE behavior and impact of the sealing sheet stiffness

To emphasize the drastic difference obtained between the isotropic Linear Elastic hypothesis and the isotropic Linear ViscoElastic hypothesis, strain–stress curves are plotted Fig. 13 for two LE cases (LE1 and LE2) and two LVE cases (LVE1 and LVE2) at point A (Fig. 8) in the bituminous mix for 40,000 cycles concerning a sealing sheet stiffness of 1,000 MPa.

In the LE cases, only tension occurs during the considered cycles. In the LVE case, the very first cycles create mainly tension but rapidly both tension and compression appear. This observation reveals that fatigue of bituminous mix during the test is not due to

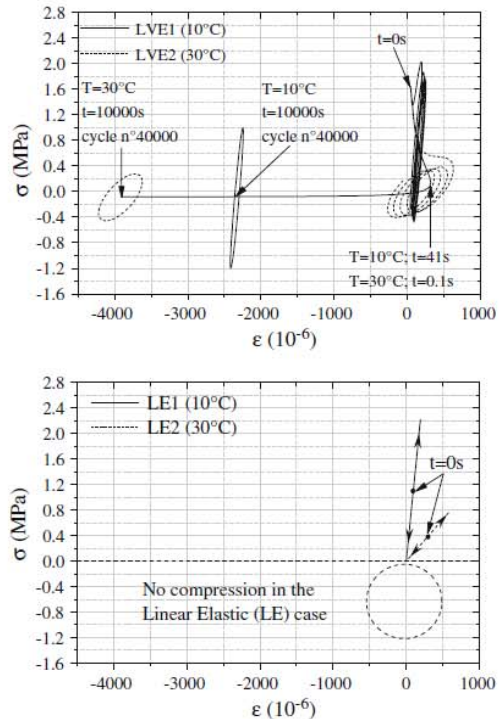


Fig. 13 Calculated stress–strain curve at point A (Fig. 8) of the bituminous wearing course for two temperatures (10 and 30°C) and a sealing sheet stiffness ( $E_{SS}$ ) of 1,000 MPa. Above: viscoelastic behavior; below: elastic behavior. For clarity of the figure, only the first four cycles and the last cycle (40,000) are plotted

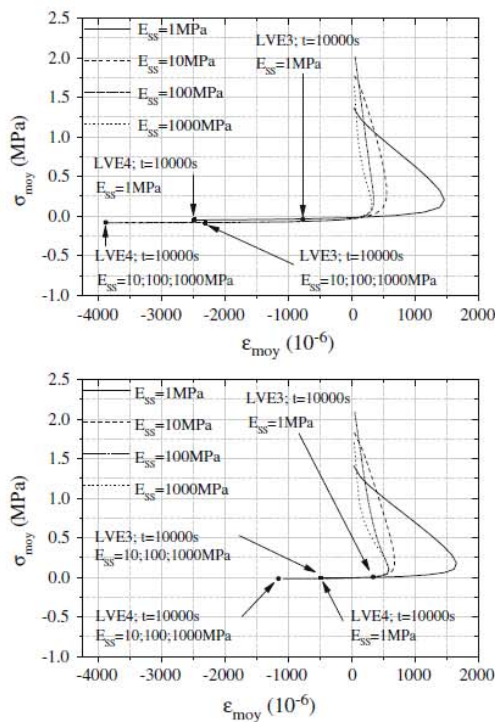


Fig. 14 Calculated average stress-average strain curve at point A (Fig. 8) of the bituminous wearing course for two temperatures (10 and 30°C) and different sealing sheet stiffnesses' ( $E_{SS}$ ). Above: viscoelastic with time-dependent  $\nu$ ; below: viscoelastic with constant  $\nu = 0.35$

cycles stress only in tension (which could induce large permanent strain) but is created by successive tension and compression phases. As a general remark, it is this type of loading (with tension and compression) that is applied during classical fatigue tests used for bituminous materials [25]. This result is at odds with the classical interpretation of the French Five-Point Bending Test, which only considers tension. Considering that bituminous mixes have very strong viscous properties, it is naturally expected that LVE calculations, which fit more correctly the real behavior, give more correct stress-strain evolution.

Figure 14 shows the evolution of the average values of  $\sigma$  versus average values of  $\epsilon$  curves for four different sealing sheet stiffnesses' considering the LVE cases with time-dependent Poisson's ratio,  $\nu(t)$  (LVE1 and LVE2), and with constant Poisson's ratio,  $\nu = 0.35$  (LVE3 and LVE4). It can be noticed that for

a fixed Poisson's ratio hypothesis and the same sealing sheet rigidity, the strain-stress curves are identical for the two temperatures (10 and 30°C). The temperature acts only on the rate at which this curve is plotted. However the chosen hypotheses for the Poisson's ratio and the sealing sheet stiffness both influence stress-strain response curve.

## 5 Conclusion and outlook

The French Five-Point Bending Fatigue Test is first presented. So far, this test is used for the choice of bituminous wearing surfacing on orthotropic steel bridges in France.

Different FEM simulations of this test were carried out considering Linear Elastic (LE) behavior and Linear ViscoElastic (LVE) behavior for the bituminous surfacing. These calculations show the great importance of taking into account the viscous effects. The interpretation of the test notably changes from solely tensile stress to alternate tensile/compressive stress at the top of the bituminous mix at point A (Fig. 8). Another observation is made concerning the relevant part of the Poisson's ratio. The response of the bituminous surfacing is different when considering constant or time-dependent Poisson's ratio, which affects the reaction of the global structure.

As a first approximation, the behavior of the bituminous sealing sheet between the steel and the surfacing is considered as isotropic linear elastic. This assumption is certainly not well verified. Due to the bituminous nature of this material, the layer should present viscous effects and also a great thermal sensitivity. A comprehensive experimental study is actually performed at ENTPE laboratory to study its behavior. However our presented numerical simulations show that the sealing sheet stiffness plays an important part in the structure response.

Concerning the French 5-Point Bending Fatigue Test, it seems that its classical interpretation needs to be reviewed to some extent. This is part of our future research developments that will include FEM calculations and comparison with in situ measurements on the Millau Viaduct—the highest bridge in the world—located in the south of France, whose overlay is a 70-mm thick dense bituminous mix.

It should be highlighted that the introduced DBN model, used to describe the LVE behavior, has





already shown its ability to take into account the much more complex behavior of bituminous mixes such as rutting, fatigue and non-linearity, which are not considered in this paper.

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## Effect of Vehicle Speed on the Millau Viaduct Response

**REFERENCE:** Pouget, Simon, Sauzéat, Cédric, Di, Hervé Benedetto, and Olard, François, "Effect of Vehicle Speed on the Millau Viaduct Response," *Journal of Testing and Evaluation*, Vol. 40, No. 7, 2012, pp. 1–7, doi:10.1520/JTE20120127. ISSN 0090-3973.

**ABSTRACT:** This paper deals with the influence of a bituminous mix surfacing on orthotropic deck bridges. These researches are part of a French national project "Orthoplus," which is briefly introduced. The approach to take into account the surfacing and to develop calculation tools is explained. First, the behavior of the different bituminous constituent materials is investigated. A linear viscoelastic modeling is proposed with a rheological model, previously developed at the Civil Engineering and Buildings Dept. ("DGCB") of the University of Lyon/ENTPE. This model is implemented in a Finite Elements code, which enables the simulation of any orthotropic structures. To validate the proposed numerical tool, the highest bridge in the world, the Millau Viaduct (France) is studied. In situ measurements were especially carried out on the Millau Viaduct. Focus is made on comparisons between experimental data and simulations results. Influence of vehicle speed on bridge response is also analyzed.

**KEYWORDS:** orthotropic steel bridges deck, bituminous materials, viscoelasticity, FEM, Millau Viaduct

### Introduction

This study is a part of a  $2.5 \times 10^6$  euro French national project called "Orthoplus: Advanced Engineering of Orthotropic Decks and Their Wearing Courses for a Global Optimization of Their Life-Cycle" [1]. The so-called Orthoplus project is led by a consortium of seven public and private partners.

The most famous example of such an orthotropic deck bridge is the Millau Viaduct (Fig. 1), known as the highest bridge (240-m high) and the longest multiple cable stayed bridge (2460-m long) in the world. The Millau Viaduct was financed and built by the Eiffage Group within the framework of a concession arrangement. Its subsidiary, "Compagnie Eiffage du Viaduc de Millau," is the concession operator of the structure for 75 years beginning at the end of 2004. The Millau Viaduct is the subject of special investigation in the "Orthoplus" project. Some in situ experiments are carried out and are used to validate the theoretical developments. In the project, three other smaller structures, with different scale, are also studied (in situ or in the laboratory) to develop and validate the calculation tools. Orthotropic is the contraction for "orthogonally anisotropic." Such bridges have a complex behavior [2–8]. Both geometry of the structure and very high flexibility of steel plates induce severe stress and strain fields in the surfacing;

hence, there are durability issues for wearing courses. The structural role of the wearing courses on orthotropic steel deck bridges is usually neglected during the design process. One objective of the "Orthoplus" project is to evaluate this role, considering the usual surfacing composed of bituminous materials.

In this paper, the linear behavior of each bituminous constituents of the deck are first determined experimentally and modeled using previous developed models, which have already proved their accuracy. These models have been implemented in commercial Finite Elements software to simulate multiple structures [9–11]. Second, one in situ orthotropic structure, the Millau Viaduct, is investigated, experimentally and with Finite Elements simulations.

### Materials Behavior and Modeling

In this paper, the structure and the constituent materials of the Millau Viaduct are considered (Fig. 1). The deck is composed of a steel plate (thickness is between 12 mm for the fast lane and 14 mm for the slow lane) reinforced with steel stiffeners. The surfacing is composed of a 3-mm-thick sealing sheet (Parafor Pont) and a 70-mm-thick bituminous surfacing layer (Orthochape). The sealing sheet made with bituminous mastic ensures the water protection of steel and the perfect bonding of bituminous surfacing on the steel.

First, the behavior of each constituent material is determined. In this paper, only the linear behavior is considered (small strain domain). No non-linearity (fatigue, permanent deformations, cracks) is taken into account. Thus, the steel is considered as isotropic linear elastic with classical parameter values ( $E = 210$  GPa and  $\nu = 0.3$ ). In the following, we focus on the bituminous materials behavior (Orthochape and Parafor Pont).

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FIG. 1—View of the Millau Viaduct (France)—the highest bridge and the longest multiple cable-stayed bridge in the world.

*Advanced Linear Viscoelastic Characterization of Bituminous Materials*

The first considered material, Orthochape, is a bituminous mix made with a continuous 0/10 mm aggregates grading and with 5.5% by weight of the aggregates of Orthoprène, a polymer-modified bitumen. Complex modulus tests in tension/compression were performed to characterize its linear viscoelastic behavior in the small strain domain ( $\epsilon < 10^{-4}$ ). The test principle is described in Fig. 2. It should be noticed that radial strain measurement ( $\epsilon_r$ ) are added to the classical axial stress ( $\sigma_z$ ) and strain measurements ( $\epsilon_z$ ) (Fig. 2). Measured sinusoidal signals (Fig. 2) are expressed in complex form (Eq 1).

$$\begin{cases} \sigma_z^* = \sigma_{0z} \cdot e^{j(\omega t)} \\ \epsilon_z^* = \epsilon_{0z} \cdot e^{j(\omega t + \phi_{\epsilon z})} \\ \epsilon_r^* = \epsilon_{0r} \cdot e^{j(\omega t + \phi_{\epsilon r})} \end{cases} \quad (1)$$

where  $\sigma_{0z}$ ,  $\epsilon_{0z}$ , and  $\epsilon_{0r}$  are the amplitude and 0,  $\phi_{\epsilon z}$ , and  $\phi_{\epsilon r}$  the phase lags of, respectively, the axial stress, axial strain, and radial strain.

The complex Young's modulus  $E^*$  and the complex Poisson's ratio  $\nu^*$  are then obtained using Eqs 2 and 3. They are defined with their norm and phase angle, respectively,  $|E^*|$  and  $\phi_E$  for the complex Young's modulus and  $|\nu^*|$  and  $\phi_\nu$  for the complex Poisson's ratio.

$$E^* = \frac{\sigma_z^*}{\epsilon_z^*} = |E^*| e^{j\phi_E} = \frac{\sigma_{0z}}{\epsilon_{0z}} \cdot e^{-j(\phi_{\epsilon z})} \quad (2)$$

$$\nu^* = -\frac{\epsilon_r^*}{\epsilon_z^*} = |\nu^*| e^{j\phi_\nu} = -\frac{\epsilon_{0r}}{\epsilon_{0z}} \cdot e^{j(\phi_{\epsilon r} - \phi_{\epsilon z})} \quad (3)$$

With the postulated hypothesis of isotropy, the 3D linear viscoelastic behavior is completely determined by  $E^*$  and  $\nu^*$  [12].

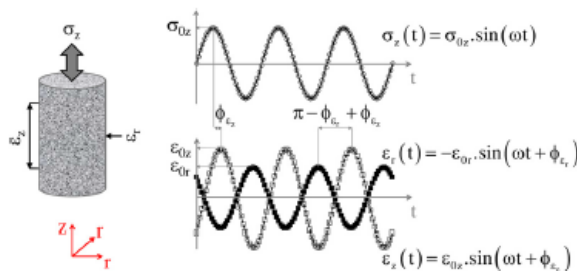


FIG. 2—Schematic explanation of the complex modulus test.

Measurements were made at nine different temperatures (from  $-30^\circ\text{C}$  to  $50^\circ\text{C}$ ), sweeping seven frequencies from 0.01 Hz to 10 Hz. From experimental data, the time temperature superposition principle (TTSP) is considered as a first approximation (some discrepancy appears at very low frequencies and/or for high temperatures, because of the effect of polymers in the Orthoprène bitumen). The complex Young's modulus  $E^*$  and the complex Poisson's ratio  $\nu^*$  results are plotted in Black diagrams in Fig. 4.

For the Parafor Pont sealing sheet, the same procedure was used. A modified great-sized rheometer prototype (the annular shear rheometer "ASR") was used [13]. The complex shear modulus was determined. The complex Young's modulus was obtained assuming a constant real value for the Poisson's ratio, 0.45. Figure 4 presents the complex modulus results in the Black diagram for the Parafor Pont material.

*Isotropic Linear Viscoelastic Modeling: The DBN Model*

A 3D isotropic linear visco-elastic (ILVE) modeling is proposed to be used for the finite-element analysis. The DBN (Di Benedetto–Neifar) model is used. This model has been developed at the University of Lyon/ENTPE. It is an attempt to describe with a unique formulation the complex elasto-visco-plastic behavior observed for different types of bituminous materials. More details on DBN model are given in Refs. 12–18. The general analogical form of the DBN model consists in an assembly in series of a linear isotropic elastic body and "n" elements having a body of EP type in parallel with a linear isotropic dashpot. It is noteworthy that the number of chosen elements "n" is a free choice for the user. If small amplitude loadings are applied, the observed behavior is linear and the obtained DBN asymptotic form is presented Fig. 3. It consists of a generalized Kelvin-Voigt model of "n" elements each having a Young's modulus  $E_i$ , a Poisson's ratio  $\nu_i$ , and a viscosity  $\eta_i$ . The calibration of the constants ( $E_i$ ,  $\nu_i$ ,

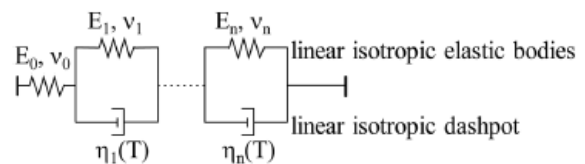


FIG. 3—Asymptotic expression in the linear domain of the DBN model (equivalent to generalized Kelvin-Voigt model).

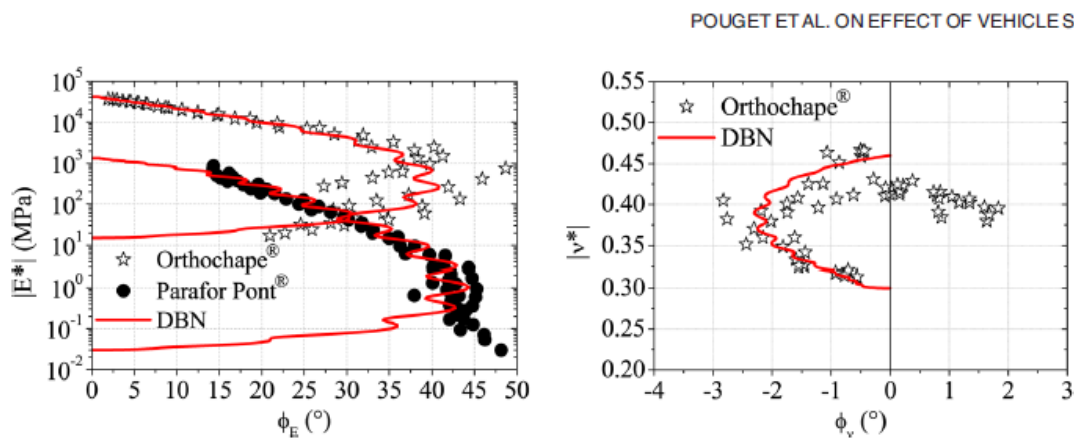


FIG. 4—Experimental Black diagrams and DBN models simulation (20 elements) for the bituminous mix Orthochape and the bituminous sealing sheet Parafor Pont plotted at a reference temperature  $T_{ref}$  equal to 10°C: (left)  $E^*$ , and (right)  $\nu^*$ .

and  $\eta_i$ ) is made from an optimization procedure in the frequency domain. Because of the lack of place, chosen constants are not given. Simulation with DBN model is also presented in Fig. 4 for Orthochape mix and for Parafor Pont sealing sheet. This linear viscoelastic model has been implemented in Finite Elements software and results from calculations on the Millau Viaduct are presented in the following.

### In Situ Measurements: The Millau Viaduct

The main orthotropic structure studied in the “Orthoplus” project is the Millau Viaduct. Some in situ experiments performed on the viaduct are described hereafter. Then some Finite Elements analyses are compared with experimental data, to show the ability of the developed calculation tools. Effects of vehicle speed is also studied.

#### Experimental Study

Experiments consisted in loading the Millau viaduct and measuring strain with a net of gauges. The slow and emergency lanes were loaded. The loading was applied by a truck and its trailer of 38.1 tons distributed on five axles (Fig. 5). Each axle was precisely weighed and the footprint of the tires was recorded. Ambient temperature was between 12.1°C and 12.3°C. Two longitudinal locations (same as for loading) were chosen for investigating the strain of the steel deck (underside). Some bi-directional gauges were also stuck below the steel deck between stiffeners 5, 6, 7, and 8. Other gauges were used under stiffeners 5, 6, 7, and 8 (Figs. 6 and 7).

Measurements were performed under two loading cases:

- Static case:  
The truck was positioned precisely. Location of the most loaded axle (second one of the tractor truck, with two twin wheels) is indicated by  $dx$  and  $dy$  (Fig. 7). This axle was located on two different longitudinal positions,  $dy$ , first, over a crossbeam ( $dy = 16.7$  m) and second in the middle of two crossbeams ( $dy = 14.6$  m). Nine different transversal

positions  $dx$  were tested (Fig. 7). One twin wheel of the axle was thus located over the stiffeners 6, 7, and 8. Transversal strain ( $\epsilon_{xx}$ ) are presented in Fig. 10 for one position of the most loaded axle, located in the middle of two crossbeams ( $dy = 14.6$  m), at transversal position  $dx = 1.77$  m. Each data point represents the average value of strain after stabilization.

- Moving case:

As for static case, location of the most loaded axle (second one of the tractor truck, with two twin wheels) is indicated by  $dx$  in transversal way (Fig. 8). Longitudinal position  $dy$  is not needed. The same nine transversal positions  $dx$  as for static case were tested. Tests were performed at two constant speeds (10 and 50 km/h) to evaluate the possible viscous effects of the surfacing on the bridge behavior. Transversal strain ( $\epsilon_{xx}$ ) are presented in Fig. 10 for one speed (10 km/h) and at transversal position  $dx = 1.75$  m.

#### Analysis With FE Calculations

Finite-element (FE) calculations are performed using “COMSOL” software in 3D to simulate experiments on the Millau viaduct. Because of the complexity and the size of the structure, some simplifications are necessary (Fig. 8):

- Six similar elements are considered longitudinally (25 m). Extremities are clamped.
- Slow and emergency lanes are modeled (4.8 m), which represents eight stiffeners.
- Crossbeams are supported on one extremity (near stiffener 1) to represent actions of the rest of the bridge structure.
- Steel plate, sealing sheet, and bituminous mix surfacing are modeled with 3D brick elements, whereas crossbeams and stiffeners are modeled with 2D shell elements. Mesh is refined around the stiffeners 6, 7, and 8, inducing  $3.12 \times 10^6$   $df$ .
- Wheel loads are modeled by rectangular surface with pressure loading (20 cm  $\times$  30 cm).

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FIG. 5—Pictures of the Millau Viaduct during the experimental measurements: (a) loading truck during weighing, (b) loading truck on the Millau Viaduct, and (c) stiffeners and crossbeam with gauges.

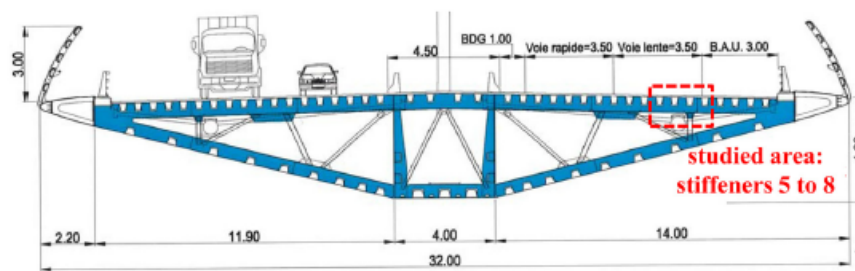


FIG. 6—Studied area of the Millau Viaduct steel deck.

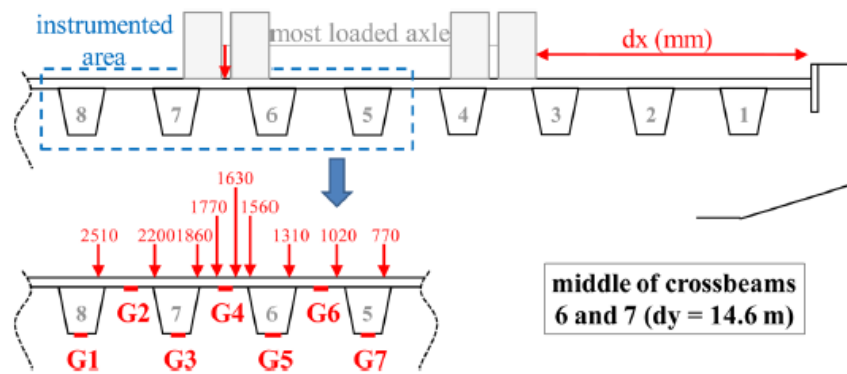


FIG. 7—Location of the load and gauges.

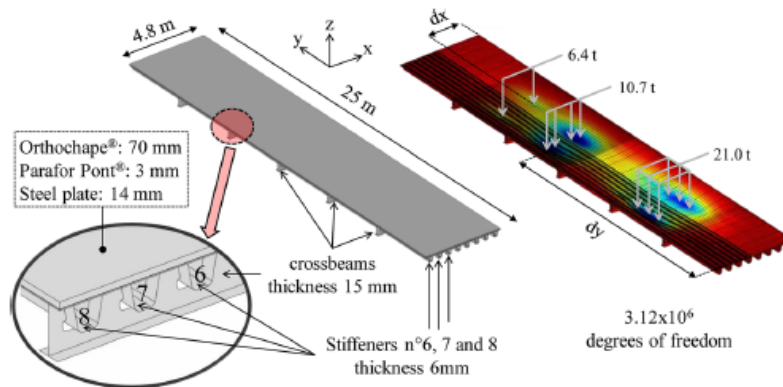


FIG. 8—Geometry, mesh, load, and vertical displacement field of the Millau Viaduct structure in the Finite Element code.

TABLE 1—Description of the different considered behaviors for bituminous materials (Orthochape and Parafor Pont) used for numerical simulations at 10°C.

Name	Behavior	$E_{Orthochape}$	$\nu_{Orthochape}$	$E_{Parafor}$	$\nu_{Parafor}$
ILE10	ILE	4290 MPa	0.41	72 MPa	0.45
ILE50	ILE	6810 MPa	0.39	113 MPa	0.45
ILE00	ILE	15 MPa	0.46	0.03 MPa	0.45
ILE0	ILE	41500 MPa	0.30	1350 MPa	0.45
ILVE	ILVE	DBN model	DBN model	DBN model	0.45

ILE, isotropic linear elastic; ILVE, isotropic linear viscoelastic.

Steel is assumed isotropic linear elastic with a Young’s modulus  $E = 210000$  MPa and a Poisson’s ratio  $\nu = 0.3$ . Influence of bituminous materials behavior constitutes the main part of this study. Different kinds of behavior are then considered for the two materials Orthochape and Parafor (Table 1):

- ILE10 and ILE50: these cases consider an isotropic linear elastic (ILE) behavior for bituminous materials. Elastic parameters are determined using values giving by DBN model ( $n = 5$  elements) at 10.5°C for the two considered speeds (10 and 50 km/h) (Fig. 9). The loading frequency induced by vehicle passing in the bituminous materials has been determined using Francken [19] and Perret [20] works. They propose a relation (Eq 4) between vehicle speed ( $V$ ) and loading frequency ( $f$ ) induced in the structural materials:

$$f[\text{Hz}] = 0.45 V[\text{km/h}] \tag{4}$$

- ILE00: this case considers an isotropic linear elastic (ILE) behavior for bituminous materials. Elastic parameters are determined from complex modulus tests for a frequency tending toward 0. The values of the two parameters, called  $E_{00}$  and  $\nu_{00}$ , are independent of temperature.
- ILE0: this case considers an isotropic linear elastic (ILE) behavior for bituminous materials. Elastic parameters are determined from complex modulus tests for a frequency

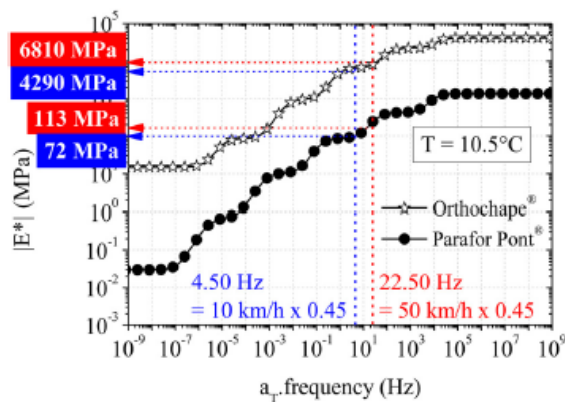


TABLE 2—Constants of DBN model ( $n = 5$ ) for the two considered materials ( $T_{ref} = 10^\circ\text{C}$ ).

(i)	Orthochape			Parafor Pont	
	$E_i$	$\eta_i$	$\nu_i$	$E_i$	$\eta_i$
0	4.15 E+04		0.30	1.35 E+03	
1	4.70 E+04	1.88 E+00	0.46	6.16 E+02	4.69 E-02
2	9.84 E+03	9.88 E+01	0.46	1.12 E+02	3.39 E+00
3	1.14 E+03	2.88 E+03	0.46	1.21 E+01	1.46 E+02
4	9.46 E+01	6.00 E+04	0.46	7.83 E-01	3.76 E+03
5	1.82 E+01	2.89 E+06	0.46	3.13 E-02	5.98 E+04

tending toward infinity. The values, called  $E_0$  and  $\nu_0$ , are independent of temperature.

- ILVE: this case considers an isotropic linear viscoelastic (ILVE) behavior for bituminous materials at the considered temperature 10.5°C. This behavior is completely defined by DBN model with  $n = 5$  elements. The number of elements (five) is chosen to limit the number of  $df$  in the FE calculation. The DBN model is calibrated from complex modulus tests (as in “Materials Behavior and Modeling” above). Constants of the DBN model are given in Table 2 for the two considered materials.

Characteristics of the five considered behaviors are given in Table 1. In this section, the tension is considered as positive. Moreover, perfect bond is assumed between layers. Some simulation results are presented in Fig. 10.

### Analysis of the Effect of Vehicle Speed on Bridge Response

Maximum strains for static and moving cases (Fig. 10) are plotted in Fig. 11 as a function of truck speed “v.” Results are given for gauge G4 located under steel plate between crossbeams 6 and 7, for an average  $dx = 1.77$  m.

This example (Fig. 11) shows result also observed for other gauges. It allows to synthesize the results (experimental and

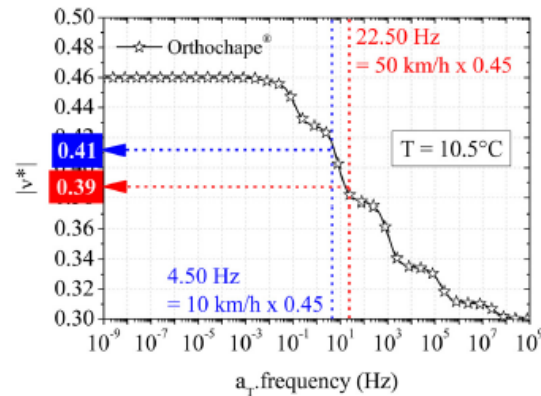


FIG. 9—Determination of elastic parameters from master curves obtained during tests on the two bituminous materials: Orthochape and Parafor Pont: (left) norm of complex modulus, and (right) norm of Poisson’s ratio.

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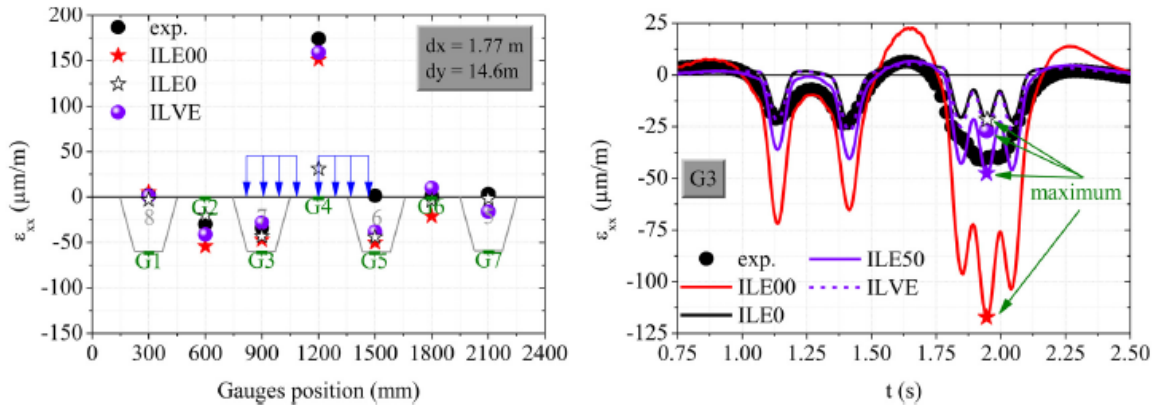


FIG. 10—Comparison between measured strains and FEM strain calculations: (left) static case for  $dx = 1.77\text{ m}$  and  $dy = 14.6\text{ m}$ , and (right) moving case at  $50\text{ km/h}$  for  $dx = 1.86\text{ m}$  and for gauge G3 located under stiffener no. 7 between two crossbeams (see Fig. 7).

calculation) obtained on the Millau Viaduct. Viscous calculations (ILVE case) allow a good approximation of strain evolution in the steel deck at any vehicle speed. However, in order not to use a heavy viscoelastic calculation, it is possible to perform “equivalent” elastic calculations following truck speed:

- In case of moving load, elastic parameters are determined using the method described in “Analysis with FE Calculations” above. Master curves plotted at a reference temperature  $T_{ref}$  corresponding to the temperature during experimental test, are used to determine Young’s modulus and Poisson’s ratio for the vehicle speed (using Eq 4).
- In case of static load, the previous method cannot be applied. Equivalent elastic parameters correspond to  $E_{00}$  and  $\nu_{00}$ , obtained at very low frequencies and/or very high temperatures during complex modulus tests should be taken. These parameters are independent of temperature.

Concerning ILE0 calculations, higher the speed is (and then frequency in the bituminous materials), more relevant the

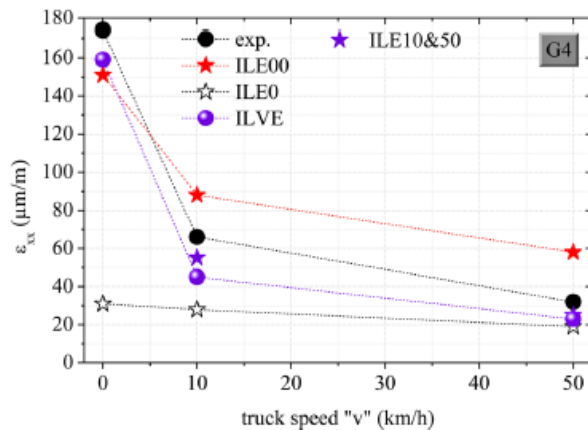


FIG. 11—Transversal strains  $\epsilon_{xx}$  measured and calculated under the steel plate (gauge 4) for different speeds (0, 10, and  $50\text{ km/h}$ ) and for  $dx$  approximately equal to  $1.77\text{ m}$ . Note: ILE00 and ILE0 do not give the same strain values for the different speeds because the truck location is slightly different.

calculation is. These results can be deduced from the analysis of viscoelastic behavior of bituminous materials.

Conclusion and Outlook

In this paper, a global approach, allowing taking into account the bituminous surfacing properties in the behavior of orthotropic structure, is explained.

First, a three-dimensional experimental characterization of bituminous materials is carried out. The linear behavior, supposed to be isotropic, is completely defined and a previously developed model (DBN) is calibrated. The ability of the model to simulate correctly the behavior of each material is shown. This model is implemented in a Finite Elements code allowing simulation of any structure.

Second, some comparisons between tests on Millau viaduct—the highest bridge in the world—and FEM numerical simulation are carried out. Experimental study on this viaduct shows the importance of taking into account viscous and thermal effects, induced by the bituminous surfacing. This requires to correctly measure and model the behavior of bituminous materials. In particular, it is shown in this paper that taking into account viscous behavior for bituminous wearing course in FE calculations allows a good approximation of experimental data at any temperature and any speed. Equivalent elastic calculations can also be performed for speeds different to zero (moving loads). In case of static load, elastic parameters  $E_{00}$  and  $\nu_{00}$ , obtained at very low frequencies and/or very high temperatures during complex modulus tests should be considered in FE calculations.

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## Annexe 3

### Formulation d'enrobés à hautes performances par un empilement granulaire optimisé et par l'emploi de bitumes modifiés en polymères (2008-2012)

Deux articles de rang A sont fournis:

- F. Olard, D. Perraton, On the optimization of the aggregate packing characteristics for the design of high-performance asphalt concretes, 4<sup>th</sup> EATA (European Asphalt Technology Association) Congress, Parma, Special Issue of Road Materials and Pavement Design, 2010. BEST PAPER AWARD.
- F. Olard, GB5 mix design: high-performance & cost-effective asphalt concretes by use of gap-graded curves & SBS modified bitumens, Annual meeting of AAPT (Association of Asphalt Paving Technologists), Austin (Texas), Special Issue of Road Materials and Pavement Design, Vol. 13, Supplement 1, pp. 234-259, 2012.

Le premier article est issu de la collaboration avec l'ETS Montréal. Le second présente le développement de la grave bitume GB5<sup>®</sup> à hautes performances, comme alternative technique et économique à la formulation traditionnelle des EME2.

L'extension internationale PCT WO2011FR50585A du brevet est enfin fournie.



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## On the optimization of the aggregate packing characteristics for the design of high-performance asphalt concretes

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*ABSTRACT. In the framework of a partnership between the European EIFFAGE Travaux Publics Group and the Ecole de Technologie Supérieure (ETS) from Montreal (Québec), some basic concepts associated with granular combinations and aggregate packing characteristics enabled the development and design of high-performance self-blocking asphalt concretes. The aggregate packing methods first developed in the field of high-performance cement concretes were successfully transposed to the field of asphalt concretes. Laboratory assessment in both laboratories of the ETS and EIFFAGE Travaux Publics consisted in the evaluation of the compacting ability using the French gyratory shear compactor (GSC), the resistance to moisture using the so-called Duriez test, the resistance to rutting thanks to the French wheel tracking tester at 60°C, the secant stiffness modulus at 15°C and the fatigue resistance at 10°C. The paper concludes that the grading analysis concepts allow the design engineer new insights into the structure of aggregates. The proposed principles can be used to develop more specific guidelines for aggregate structure selection of high-performance asphalt concretes.*

*KEYWORDS: mix design, optimal aggregate packing, aggregate interlock, gap-graded asphalts, self-blocking asphalts, high-performance asphalt concretes (HPAs).*

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### 1. Introduction

Controlling the volumetrics and the compactability of asphalt concretes is the first step of any mix design procedure. Apart from the binder-linked considerations, as the aggregate component represents 90 to 95% of the weight of asphalt mixture, predicting and controlling the packing properties of aggregates is thus of prime importance. Broadly speaking, the aggregate packing characteristics are mainly influenced by the four following parameters (Caquot, 1937; Baron, 1982, de Larrard *et al.*, 1988, 1994a, 1994b, 2002; Corté & Di Benedetto, 2004):

- Gradation (continuously-graded, gap-graded, etc.)
- Shape (flat & elongated, cubical, round)
- Surface micro-texture (smooth, rough)
- Type & amount of compaction effort (static pressure, impact or shearing)

This paper focuses in particular on the first parameter (gradation) by specifically optimizing the combination of fine and coarse fractions in such a way that results in an interactive network of coarse particles in asphalt concrete, providing indirectly the strongest mix resistance (Roque *et al.*, 1997; Kim *et al.*, 2008).

### 2. Theoretical background on aggregate packing

In a non-exhaustive way, two basic methods, respectively developed in the fields of asphalt concretes and cement concretes, are detailed in the two next sub-sections (2.1 and 2.2) with a view to introduce the work of this study.

#### 2.1. The "Bailey method" developed in the field of asphalt concretes

In the field of asphalts, in the early 1980s, Bailey, now retired from the Illinois Department of Transportation, developed a series of analytical procedures to evaluate mixes being proposed by contractors in the district where he was the Chief Materials Engineer. He used this new analytical tool to predict mixture volumetrics and to offer suggestions on how to make the necessary gradation changes to meet the volumetric requirements. As the use of this technique began to spread across Illinois and Indiana, it became known as the "Bailey Method". The procedure finally came to the attention of Transportation Research Board's committees on HMA mixtures, which published a technical circular in October 2002, entitled, *Bailey Method for Gradation Selection in Hot-Mix Asphalt Mixture Design* (Varvik *et al.*, 2002).

This is an approach to blending aggregates that provides aggregate interlock and a balanced gradation. First, a new relevant definition of "fines" and "stones" (coarse aggregates) is introduced: fine aggregates are those particles that can fill the voids created by the stones and division is not related to the conventional No. 4 sieve

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(4.75 mm) or European equivalent 4 mm sieve opening (commonly referred to as the "split"). The sieve that separates the coarse and fine aggregate is named the Primary Control Sieve (PCS), depending on the so-called Nominal Maximum Particle Size (NMPS) of the aggregate blend. According to Bailey, the PCS is defined as 0.22 of the NMPS based on two and three dimensional analysis of the packing of different shaped particles (Fig. 1). The value of 0.22 represents the coarse-fine diameter ratio enabling the larger fine aggregate particle to fill the voids created by the coarse aggregate particles. The 0.22 factor is the average condition of many different packing configurations. Besides, the aggregate blend below the PCS (fines) is divided into coarse and fine portions –using the same ratio of 0.22–. Each of these three portions is then evaluated so that the final blend is optimized (Fig. 2).

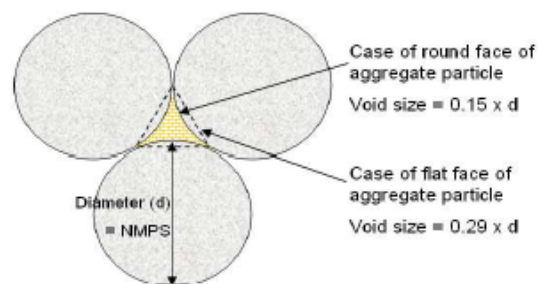


Figure 1. Two-dimensional aggregate packing model by Bailey (Aurilio et al., 2005). NMPS: Nominal Maximum Particle Size.

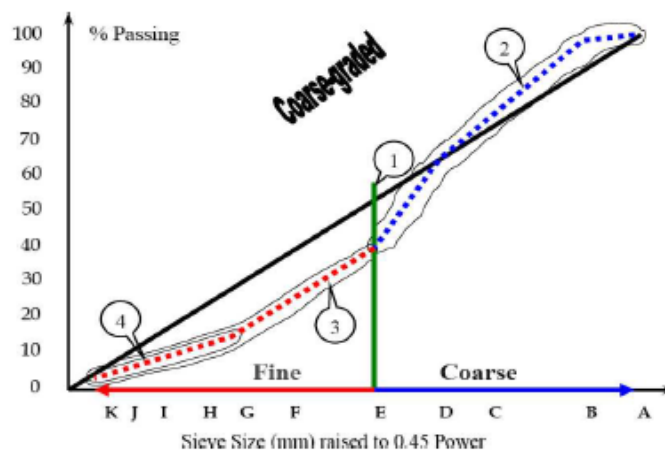


Figure 2. The Bailey method relying on a new definition of what is "fine" or "coarse", depicted on a typical gradation curve for a coarse-graded mix. The so-called Primary Control Sieve (PCS) is defined as 0.22 of the Nominal Maximum Particle Size of the aggregate blend (Vavrik et al., 2001).

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### *2.2. Basic concepts of aggregate packing characteristics first developed in the field of cement concretes*

Likewise, the proportioning of fine and coarse aggregates in the cement concrete has an important effect on the properties of both fresh and hardened concrete. Some guides provide a means to determine the optimal gradation of fine and coarse aggregates for use in the concrete mix using a set of tables. The tables are either based on very empirical approaches or on computer models for the theoretical packing of spherical particles which takes into account their size and specific gravity. In the field of cement concrete, the use of these tables helps produce a more workable mix and a better consolidated hardened concrete with decreased permeability and improved durability. Such tables are for instance given in the SHRP-C-334 guide (Andersen & Johansen, 1993).

With the ever-continuing development of computers, the packing problems of real systems became a challenging subject for engineers (Stroeven & Stroeven, 1999). Such algorithms are usually based on the modeling of the movement of particles (represented as spheres or ellipsoids) due to rolling or sliding under the compaction gradient. A comprehensive survey of contemporary packing algorithms was recently presented by Jia and Williams (2001). Modern modeling approaches include better insight into the natural packing process and even full-scale modeling of the particulate systems. The contribution of additional factors (such as friction and deformation) acting at the contact points was found to be essential for modeling the dynamic processes, involving particulate materials. To realize such algorithms, each movement of a particle requires the solution of the relevant differential equation; this procedure slows down the calculation process.

Quite apart from such refined packing models and algorithms, some basic concepts of aggregate packing characteristics were first developed by Caquot in 1937, then by contemporary researchers since the 1970's. A comprehensive state-of-the-art of such both simple and effective basics was recently presented by Perraton *et al.* (2007), trying to transpose those concepts in the field of asphalt mix design. The following sub-sections are partially drawn from this paper.

#### *2.2.1. Basic notions associated with granular combinations*

When filling a container with an aggregate or granular mix, just a fraction of the volume is occupied by the particles and the other part is composed of interstices. It is worthwhile to remark that for an infinite medium, the void index ( $e$ ) of an aggregate composed of one-dimensional particles remains, for all practical purposes, independent of particle size (Ben Aïm, 1967, 1970; Cumberland & Crawford, 1987).

When studying the porosity of mixes composed of two aggregates with differing yet one-dimensional individual sizes, Caquot (1937) first highlighted the importance

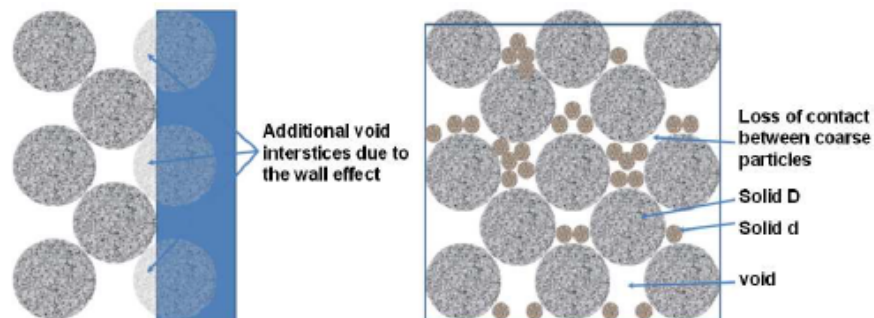
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of two types of interparticle interaction on the void index: the so-called "wall effect" and "interference effect" –the latter is also called "loosening effect"–.

The "wall" effect is tied to the interaction between particles and any type of wall (pipe, formwork, *etc.*) placed in contact with the granular mass. Let's take the case of a uniform, two-aggregate mix that only differs by its average particle dimension, *i.e.* one for the coarse aggregate particles and another for the fines. When adding a few coarse particles into an infinite volume of fines, the void index of the blend is reduced. Nevertheless, the coarse particles locally, at the interface, disturb the arrangement of fines whose porosity is increasing (*i.e.* an increasing void index). This local porosity increase is proportional to the particle surface area of the incorporated coarse aggregate (Caquot, 1937; Chanvillard, 1999). Figure 3 (left) and Figure 4 indicate the wall effect on the void index of a granular combination when raising the coarse fraction within the fine aggregate of a binary mix.

In continuing to increase the part of coarse particles within the fines at a certain point, a specific quantity of small particles winds up entrapped in the interstices delimited by the coarse particles. Therefore, outside of the wall effect, the fine aggregate void index increases due to interference: the arrangement of fine particles will depend not just on the surface areas of coarse particle walls (the wall effect), but also on the actual layout of these particles, *i.e.* the shape of their interstices.

The "interference" (or "loosening") notion can be illustrated by focusing on the effect induced by introducing a few fine particles into an infinite volume of coarse particles. As the amount of fines increases, at some point the coarse particles are forced apart by means of loosening, thus modifying their spatial configuration: interference is present. Figures 3 (right) and 4 also display the interference effect on the void index of a binary granular combination.



**Figure 3.** (Left) Schematic of the wall effect (case of the contact of fine particles at the surface of a coarse particle). (Right) Schematic of the loosening effect (loss of stone-to-stone contact due to the all too high content of fine particles).



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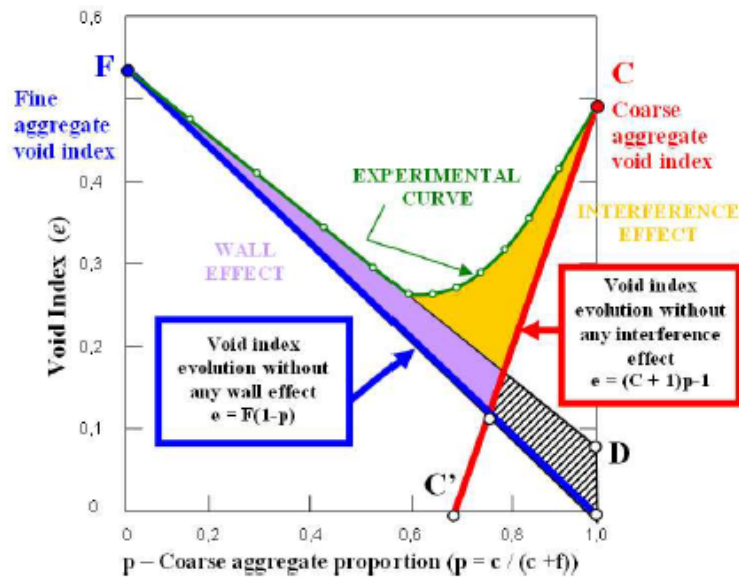


Figure 4. Schematic of the wall and interference effects according to Powers (1968).  $f$  and  $c$  are respectively the solid volumes of the fine and the coarse aggregates ( $f+c=1$ );  $F$  (resp.  $C$ )= void index of the fine (resp. coarse) aggregates.

If the average particle dimension of the fines ( $d_{\text{FINE}}$ ) is small enough compared to that of the coarse particles ( $d_{\text{COARSE}}$ ) (e.g. when the  $d_{\text{FINE}}/d_{\text{COARSE}}$  ratio is less than 0.2), the wall effect is linear and satisfies the superposition principle: the effect of two walls amounts to the sum of effects of each wall taken individually (Baron & Sauterey, 1982). In contrast, the interference/loosening effect is never linear and consequently difficult to frame simplistically (Baron & Sauterey, 1982).

### 2.2.2. Evolution in aggregate porosity Vs. average particle dimension

Powers (1968) noted that Furnas had formerly evidenced the dependence of the evolution in void index ( $e$ ) Vs. coarse aggregate portion in a one-dimensional binary combination on the ratio of average particle sizes. Figure 5 –from Furnas' work–, reveals that as the ratio of average fine aggregate dimension-to-average coarse aggregate dimension rises, interaction effects become more significant as well.

In order to minimize interactions of the intermediate particle fraction on the coarsest particles in the asphalt concrete, it is crucial to limit both their size and amount and fill voids instead by a higher fraction of fines. Let's point out the experimental evidence showing that the minimum void volume of a granular mix depends first and foremost on the ratio of the coarsest particle dimension to the finest and that the range of intermediate dimensions exerts only a slight influence (Baron, 1982).

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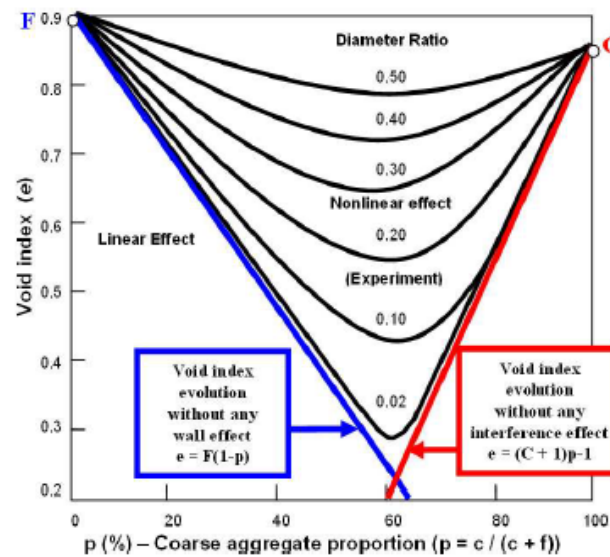


Figure 5. Evolution in the Furnas void index ( $e$ ), as established on a binary combination of coarse ( $c$ ) and fine ( $f$ ) aggregates Vs. the proportion of coarse aggregate in the combination for various particle diameter ratios ( $d_{\text{COARSE}}/d_{\text{FINE}}$ ) (according to Powers (1968), Oger (1987) and Perraton et al. (2007)).

Furthermore, Figure 5 points out that the so-called PCS value (0.22) used in the Bailey method is not large enough to prevent any "interparticle" interaction effect between coarse and fine aggregates, even with referring to the new relevant definition of "fines" and "stones" introduced by Bailey.

### 2.2.3. Mix of an extremely fine aggregate with a coarse aggregate

For a situation in which one aggregate is very fine in comparison with the other ( $d_{\text{FINE}}/d_{\text{COARSE}} \sim 0.008$ ), Baron (1982) proposed describing the void index variation of a mix by means of three straight lines (Fig. 6). This case proves insightful since it corresponds to the mix of filler ( $d_{\text{FILLER}} < 80 \mu\text{m}$ ) with a uniform granular skeleton.

Baron defined two thresholds,  $p_X$  and  $p_T$ , which indicate the critical concentrations that allow eliminating interference effects. Within a binary mix (containing coarse and fine particles), the threshold  $p_X$  corresponds to the maximum coarse aggregate concentration that can be combined with fine aggregate without adversely altering the fine aggregate arrangement, whereas threshold  $p_T$  allows establishing the maximum fine aggregate concentration (*i.e.*  $1-p_T$ ) for combination with coarse aggregate so as not to interfere with the coarse particle layout.

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Depending on whether the granular mixture has a high ( $p < p_x$ ), medium ( $p_x < p < p_r$ ) or low ( $p > p_r$ ) content of fine aggregate, the void index variation can be defined according to three distinct laws as shown in Figure 6:

- High content of fines in the mix,  $p < p_x$ :

$$\Rightarrow e = F(1 - p) + Dp \quad [1]$$

where  $F$  is the void index of fines and  $D$  a coefficient of the wall effect (Fig. 6).

- Low content of fines in the mix,  $p > p_r$ :

$$\Rightarrow e = (C + 1)p - 1 \quad [2]$$

where  $C$  is the void index of the coarse particles (Fig. 6).

- Medium content of fines in the mix,  $p_x < p < p_r$ :

$$\Rightarrow e = Ep \quad [3]$$

where  $E$  is a coefficient without any simple physical significance.

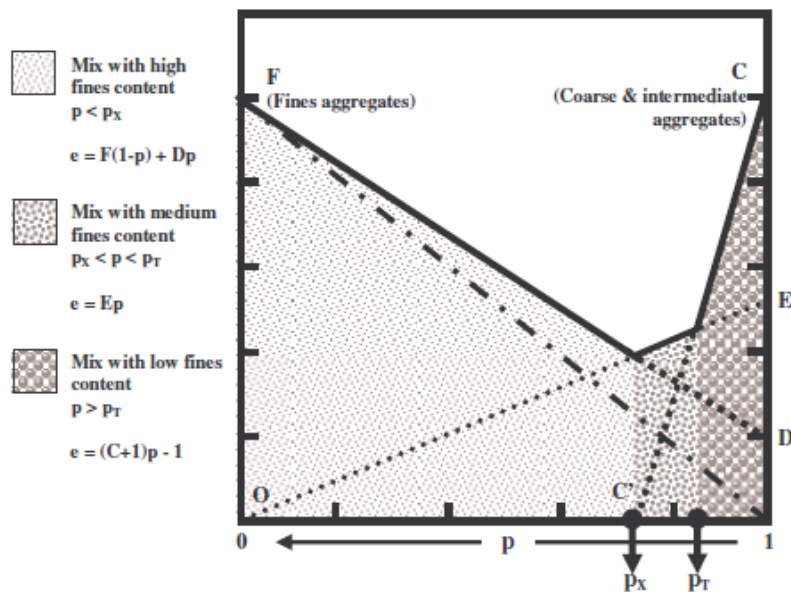


Figure 6. Void index variation ( $e$ ) in the case of a two-aggregate mixture, one of which is very fine compared to the other (according to Baron (1982)).

#### 2.2.4. Transposition of the "Baron's approach" to the design of Stone-Matrix Asphalts with an optimal coarse aggregate packing according to Perraton (2007)

By applying the concepts initially developed by Baron for designing high-performance cement concretes, Perraton and his colleagues from the Ecole de Technologie Supérieure (University of Quebec) have recently succeeded in illustrating that the lab performances of some SMA mixes may be substantially enhanced by means of a maximized coarse aggregate packing (Perraton *et al.*, 2007).

The proposed mix design method was then applied to producing the so-called "SMA-Cpack" (acronym for SMA with an optimal coarse aggregate packing) mixes with various NMPS values using materials available in the Montreal area. Figure 7 shows a schematic diagram of the macrostructure in asphalt mix section cuts obtained using digital imagery. It may be observed that the coarse particle content is high for each of the SMA-Cpack mixes produced.

First, the initial set of lab results suggests a potential saving of nearly 15% binder since the maximized coarse aggregate packing brings about a reduced void content in the granular skeleton. Second, based on Perraton's results, the compacting ability and resistance to rutting of such mixtures are found to be very good.

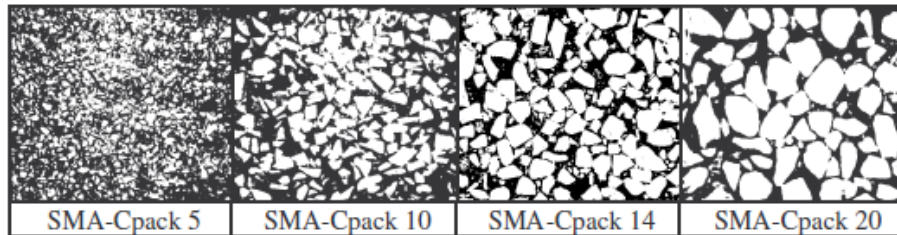


Figure 7. Illustration of the coarse aggregate proportion in the SMA-Cpack mixes (Perraton *et al.*, 2007).

### 3. Objective of the present study

In the framework of a partnership between EIFFAGE Travaux Publics and the Ecole de Technologie Supérieure (ETS), the previous concepts associated with granular combinations and packing characteristics were used in the EIFFAGE research centre in Lyon (France). The Baron's approach for optimal aggregate packing, first developed in the field of high-performance cement concretes (1982), was transposed to the field of asphalt concretes. The main goal was to evaluate the relevancy of such an approach in the asphalt field, especially with typical French

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aggregates. The underlying questions were: can we develop high-performance dense asphalts with multiple-gap grading mainly thanks to aggregate packing optimization? And, likewise, can we use more specific guidelines for aggregate structure selection?

After the publication in the French *Bulletin de Liaison des Ponts et Chaussées* of the first encouraging results obtained by Perraton *et al.* (2007), Eiffage Travaux Publics has launched a large experimental campaign at its research centre, including the evaluation of moisture resistance, stiffness modulus and fatigue resistance as well. Lab assessment consisted in the assessment of the compacting ability using the French gyratory shear compactor (GSC), the moisture resistance using the Duriez test, the rutting resistance thanks to the French wheel tracking tester (FWTT) at 60°C, the stiffness modulus at 15°C and finally the fatigue resistance at 10°C.

### 4. The proposed way of aggregate packing optimization

To make the simplest possible selection of an optimal granular combination with a maximized coarse fraction, it is proposed herein to separate the granular skeleton assembly into three phases: fine, intermediate and coarse particles. The latter actually constitute the granular fraction that provides for continuity in handling the force transfer within the mix macrostructure, otherwise known as coarse-on-coarse contact. These particles on their own serve to define aggregate NMPS.

To enhance compactability, this methodology proposes monitoring the choice of granular combination so as to overcome the loosening effect from the intermediate fraction on the coarse fraction arrangement. In particular, the intermediate particle dimension must be held below a certain critical value. With respect to Furnas' work (Figure 5), it is advised to limit *a priori* the ratio of intermediate particle diameter ( $d_{INT}$ ) to coarse particle diameter ( $d_{COARSE}$ ) to 20% ( $d_{INT} \leq 0.20 d_{COARSE}$ ) (see the average particle dimension of the respective fractions in section 4). Likewise,  $d_{FINE} \leq 0.20 d_{INT}$ .

Using the French gyratory shear compactor (GSC) on aggregates only –without any bitumen–, the respective void index of coarse, intermediate and fine aggregate particles is first determined using the GSC after 20 gyrations. Indeed, after 100 gyrations, attrition, segregation and abrasion may be observed (Figure 8).

Depending on the NMPS of the designed mix and thus on the number of used granular fraction ( $n$ ), the optimization sequence is performed during  $n-1$  steps. For instance, in the case of a 3-fraction mix with fine, intermediate and coarse aggregates, the optimization sequence is performed during two steps:

- Step 1: granular optimisation of the intermediate aggregate-coarse aggregate blend (*i.e.* determination of the corresponding optimal ratio according to Baron's approach illustrated in Fig. 6).
- Step 2: granular optimisation of the blend between the previous optimized aggregate blend and the fine aggregates (same methodology, described in Fig. 6).

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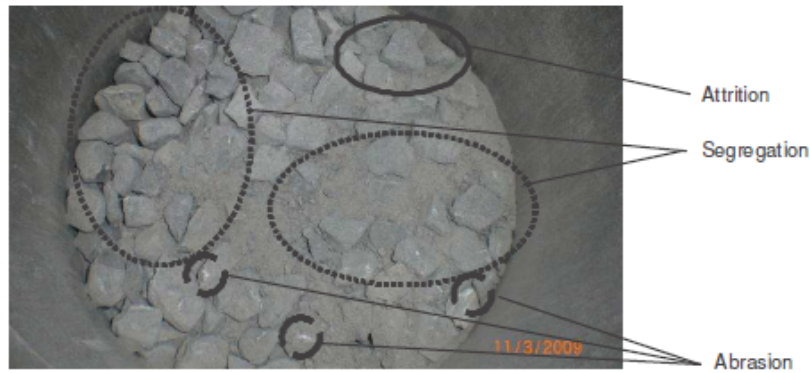


Figure 8. Example of the attrition, segregation and abrasion phenomena observed at 100 gyrations using the gyratory shear compactor (GSC).

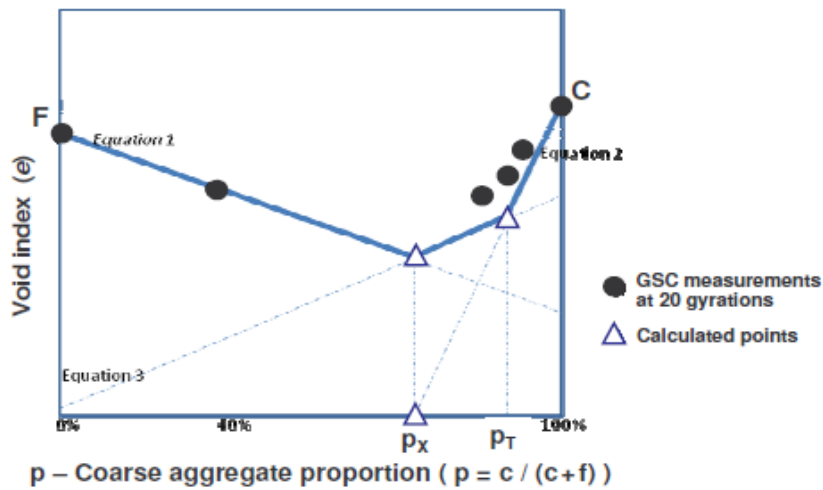


Figure 9. Experimental determination of the two thresholds  $p_X$  and  $p_T$ .

For each step ( $i$ ), the only slight difference with the original Baron's approach is that a sensibility study is realized around the thresholds  $p_T$  (Figure 9). Two additional points are performed around the  $p_T$  value ( $p_T \pm 3\%$ ). Moreover, for each step ( $i$ ), Equation 1 is defined by carrying out two GSC tests: the first one with  $p=0\%$ , the second one with  $p=40\%$ . As illustrated in Figure 9, six GSC tests are performed for each step ( $i$ ).

Eventually, in the case of a 3-fraction mix with fine, intermediate and coarse aggregate particles, the granular optimization is performed with a series of twelve GSC tests on this ternary configuration.

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## 5. Materials

Because of the great number of aggregate combinations that were tested in the framework of this lab study, only one aggregate nature was used. The diorite crushed aggregate fractions (0/2, 0/4 and 10/14 mm) come from the "La Noubleau" quarry in France. Besides, the added limestone filler comes from the "S<sup>t</sup> Hilaire" quarry in France.

Moreover, some reclaimed asphalt pavement (RAP) aggregates coming from the "Touraine Enrobés" asphalt plant (EIFFAGE Travaux Publics Centre Ile-de-France) were used. An analysis of the recovered aged binder of the RAP showed that the content, the penetration and the softening point Ring and Ball were respectively 5.1%, 10 (mm/10) and 65.0°C.

Table 1 and Figure 10 give the gradation curves of each tested granular fraction.

Apart from the average dimension of added filler being determined thanks to the Coulter<sup>®</sup> particle size analyzer, the average particle dimension of the other fractions was determined by sieve passing. These values are given in Table 1.

Sieve (mm)	Passing (%)				
	Added filler	0/2	0/4	10/14	RAP
16				100	100
14				93	97
12.5				77	89
10				22	72
8				5	63
6.3				1.7	55
5			100	0.8	50
4			96	0.4	45
3.15		100	85	0.3	41
2		97	54	0.3	32
1		68	37	0.2	23
0.5		45	25	0.2	17
0.315		35	20	0.2	13
0.25	100	31	18	0.2	10
0.125	94	22	13	0.2	10
0.08	83	17	10	0.2	9
<b>Average particle dimension</b>					
<b>Diameter (mm)</b>	0.025	0.6	1.9	11.5	5

Table 1. Passing percentage for each tested granular fraction.

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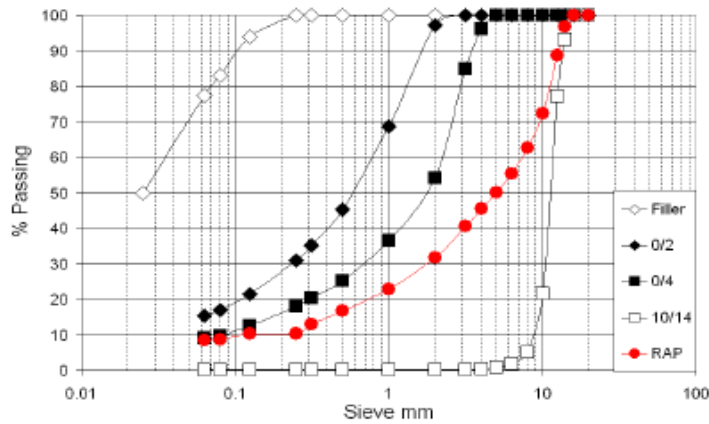


Figure 10. Gradation curves for each tested granular fraction.

binder	Pen 25°C (mm/10)	Softening Point R&B (°C)	Fraass (°C)
10/20	15	62.0	-10
35/50	38	53.5	-15
aged RAP binder	10	65.0	-

Table 2. Conventional results on the different binders.

For asphalt concretes prepared in laboratory, two pure paraffinic bitumens coming from the same crude and same refinery were investigated: 10/20 and 35/50 pen grade bitumens. Table 2 presents the results of the conventional tests (Penetration at 25°C and Softening Point Ring and Ball, Fraass brittle point) initially performed on these two different binders.

## 6. Description of the tests used for characterizing asphalt concretes

Many laboratory tests were conducted on asphalt concretes, including:

- Compactability, measured from the gyratory shear compactor (GSC), following the requirements of the standard NF EN 12697-31. This test gives a good idea of the job-site density values, according to course thickness. Conducted ahead of the other mechanical tests, this test is used to make a preliminary selection or screening of mixes, and for optimizing the asphalt mix composition (Harman *et al.*, 2002).

- Water resistance, measured from the so-called Duriez test (NF EN 12697-12) which consists of direct compression test on two sets of six cylindrical samples, one



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set of six samples tested after conditioning in water. If the mean ratio of the results after and before conditioning is above a certain value, the material is deemed to be acceptable. This ratio is the French counterpart of the TSR (Tensile Strength Ratio) value with Marshall samples.

- Resistance to rutting at 60°C, characterized with the French wheel tracking tester (FWTT) in accordance with NF EN 12697-22 (rectangular specimens subjected to repeated passes at 1Hz of a wheel fitted with a tyre, inducing permanent deformation).

- Secant stiffness modulus measured at 15°C-0.02s, following the requirements of the standard NF EN 12697-26 (strain-controlled test on cylindrical specimens).

- Complex stiffness modulus and fatigue resistance at 10°C-25Hz, following the the NF EN 12697-24 requirements (controlled-strain test on trapezoidal specimens, see Figure 11). The criterion that is used in this paper to determine the mix fatigue life is the classical one, referenced as  $N_{150}$ . It corresponds to the number of cycles for which, either the complex modulus decreases of 50% of its initial value, or a sudden failure occurs (before 50% decrease of modulus). The value of the strain amplitude leading to failure at one million cycles is hereafter called  $\epsilon_6$ . This parameter is in particular used in the French design method (SETRA-LCPC, 1997).



Figure 11. *Complex modulus test and fatigue test configuration on trapezoidal specimens (NF EN 12697-24).*

#### 7. Aggregate packing optimization results

Figure 12 (resp. Figure 13) illustrates the iterative aggregate packing optimization of a ternary 10/14-0/2-filler (resp. a quaternary 10/14-0/4-0/2-filler) aggregate combination, by using the gyratory shear compactor (GSC) on aggregates only –without any bitumen– as already detailed in section 4.

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Unlike Figure 12, Figure 13 shows a noticeable difference between the calculated  $e(p_X)$  and  $e(p_T)$ , from one hand, and the corresponding experimental data, from the other hand. The reason for such a difference mainly lies in the fact that in this study  $d_{0/2}/d_{10/14} = 0.052 < d_{0/4}/d_{10/14} = 0.165$ , which brings about somewhat interference in the case of the 0/4-10/14 binary aggregate composition.

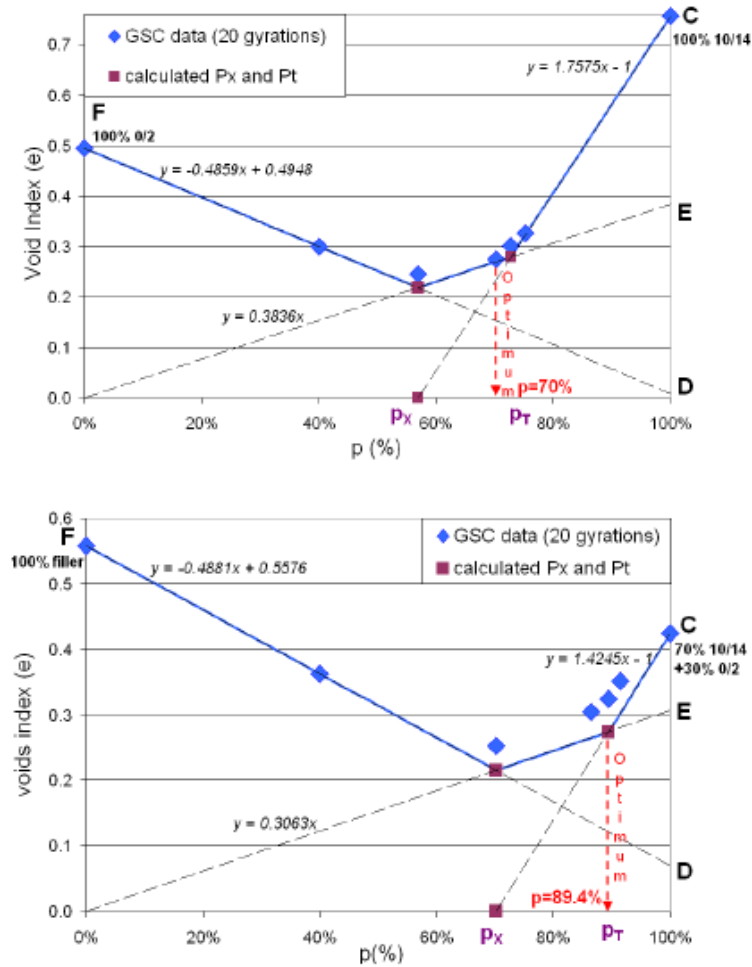


Figure 12. Two-step iterative optimization of the 10/14-0/2-filler ternary aggregate blend.

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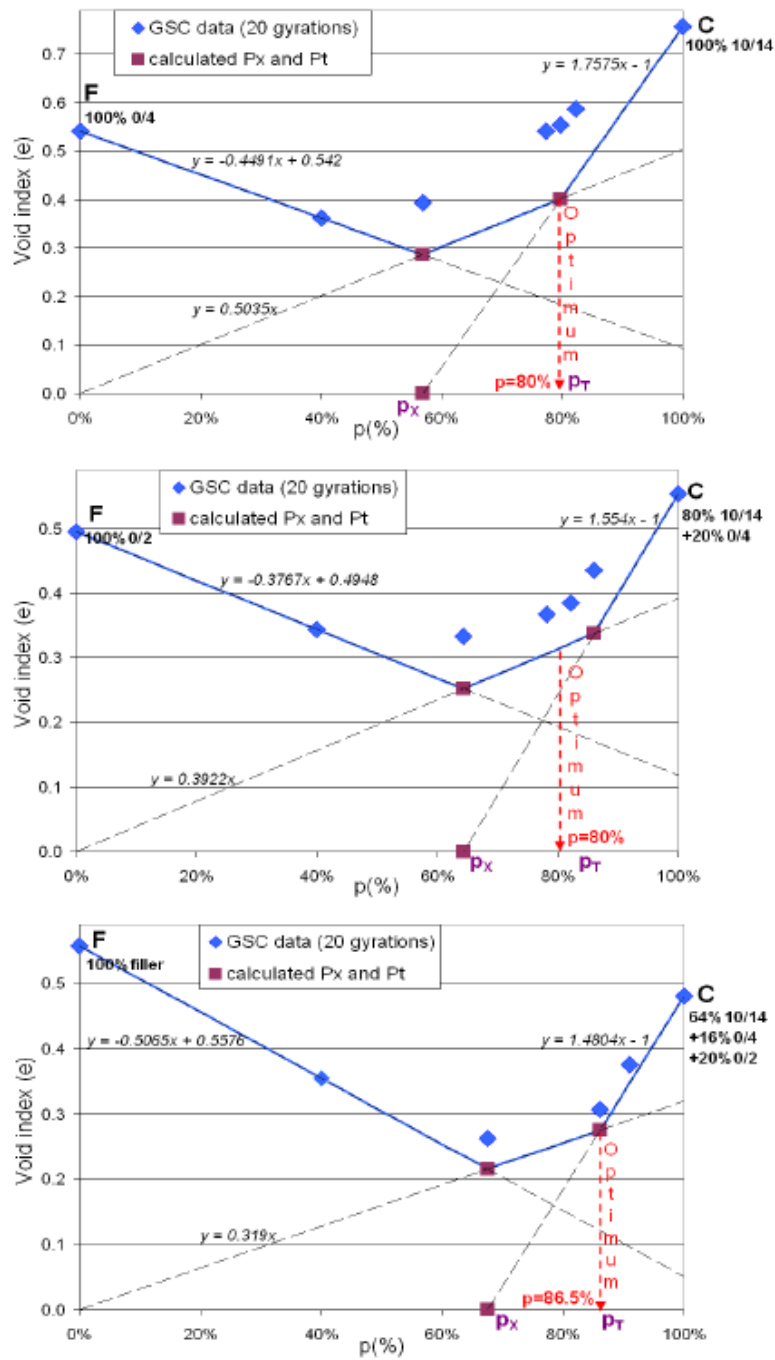


Figure 13. Three-step iterative optimization of the 10/14-0/4-0/2-filler quaternary aggregate blend.

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For both 10/14-0/2-filler and 10/14-0/4-0/2-filler configurations, considering the packing optimization with the 0/2 fraction as "fines", the optimal  $p$  value (optimal proportion of coarse particles) was arbitrarily fixed to a slightly lower value than  $p_T$ . Indeed, the following and last stage of aggregate packing optimization with filler generally led to all too high filler contents (from both economic and practical outlook on the asphalt plant), far beyond 10%.

Insofar the 10/14-0/2-filler ternary combination is concerned, the aggregate packing optimization consists of two sets of GSC measurements at 20 gyrations:

- the first set of GSC tests is performed in order to determine the optimal 10/14-0/2 binary blend (in this case,  $p=70\%$  (i.e. 70% 10/14 and 30% 0/2), cf. Fig 12);
- the second set of GSC tests is performed to determine the optimal 10/14-0/2-filler ternary blend, which is considered as a binary blend: the previous 70% 10/14-30% 0/2 blend is considered as the "coarse fraction", whereas the filler is considered as the "fine fraction". In this case, the optimal coarse ratio is found to be  $p_T=89.4\%$ .

Therefore the optimal 10/14-0/2-filler ternary blend is the following:

- 10/14 content: 62.58% ( $=0.7*0.894$ )
- 0/2 content: 26.70% ( $=(1-0.7)*0.894=0.3*0.894$ )
- added filler content: 10.6% ( $=1-0.894$ )

As regards the 10/14-0/4-0/2-filler quaternary blend (Fig. 13), the packing optimization consists of three sets of GSC measurements at 20 gyrations. The optimal 10/14-0/4-0/2-filler quaternary blend is the following:

- 10/14 content: 55.3% ( $=0.865*0.64$ )
- 0/4 content: 13.9% ( $=0.865*0.16$ )
- 0/2 content: 17.3% ( $=(0.865*0.2)$ )
- added filler content: 13.5% ( $=1-0.865$ )

Because of a slight segregation observed for the 10/14-0/2-filler ternary blend when preparing and removing the samples from the GSC molds, the 10/14-0/4-0/2-filler quaternary blend was first preferred to start the laboratory campaign on asphalt mixes. The results of such a performant study are presented in the next section.

For both practical and economical reasons, two additional gradations –very close to the "optimal" one– with only 5% filler content were studied: the first one without RAP, the second one with 10% RAP. Indeed, the metering of 10% added filler is not convenient on most batch plants and is even impossible on the drum mix plants that are currently used. Table 3 and Figure 14 give the gradation curves of each tested asphalt concrete. Figure 14 shows in particular that the passing of the different HPA aggregate gradations at the 2-mm sieve is above the SMA envelope (NF EN 13108-5, 2006), which is indirectly due to the 4/10 gap gradation.

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Sieve (mm)	Passing (%)			
	Reference Grave Bitume GB2 0/14	HPA 0/14 with 10% added filler (4/10 gap-graded)	HPA 0/14 with 5% added filler (4/10 gap-graded)	HPA 0/14 with 5% added filler (4/10 gap-graded) + 10% RAP
20	100	100	100	100
16	96	96	96	96
14	94	92	92	93
12.5	88	83	83	84
10	67	51	51	54
8	57	42	42	44
6.3	50	40	40	42
5	45	39	39	41
4	42	38	38	39
3.15	40	37	37	37
2	34	32	32	32
1	25	25	24	24
0.5	17	20	18	18
0.315	14	18	15	15
0.25	13	17	14	14
0.125	9	14	11	11
0.08	8	12	9	9

Table 3. Passing percentage for each tested asphalt mix. "HPA" stands for "high-performance asphalt".

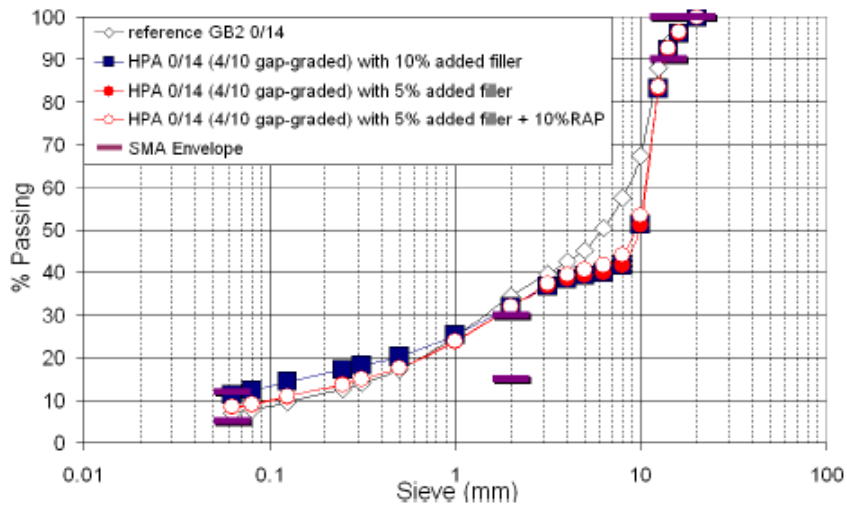


Figure 14. Gradation curves for the optimal 10/14-0/4-0/2-filler quaternary aggregate blends Vs that of the reference "Grave Bitume" GB2. In the caption, "HPA" stands for "high-performance asphalt".

### 8. Performantial characterization of the asphalt concretes produced with such optimal coarse aggregate packing and related discussion

Richness modulus ( $k$ ) is essentially the ratio of bitumen to surface area of aggregate and thus an estimation of binder film thickness. For more details, the reader can refer for instance to Fontana *et al.* (1998) or Corté & Di Benedetto (2004). This parameter, of primary importance in France, was kept constant and equal to 2.6, whatever the tested asphalt concrete (binder content in volume is also given in Table 4). Note that a value of 2.6 for richness modulus can be considered as low in both France and Québec.

Formula					GSC	Duriez Test		Rut Depth 10,000 cycles	E 15°C-0.02s (MPa)	$\epsilon_6$ 10°C-25Hz ( $\mu$ strains)
% added filler	k	%vol. binder	%RAP	%voids at 100 gyrations	R (MPa)	Moist. Res. (%)				
Specifications for "Grave Bitume 2" (GB2)					<10%	-	>70%	<10%	>9000	>80
GB2	-	2.6	10.0	0	9.7	10.1	93	4.1	14,300 at 4.1% void	to be done (tbd)
HPA	10		10.6	0	5.9	13.6	87	10.2	16,400 at 2.6% void	tbd
	5		10.0	0	5.9	11.8	83	5.1	15,900 at 2.7% void	tbd
			10.0	10	7.4	12.1	91	4.1	16,400 at 2.7% void	tbd

**Table 4.** Mechanical performances of the three developed high-performance asphalts (HPAs) compared to the reference Grave Bitume GB2 material (constant richness modulus:  $k=2.6$ ).

Table 4 presents the mechanical assessment of the developed high-performance asphalts (HPAs) compared to the reference Grave Bitume GB2 material:

- Compacting ability, measured from the gyratory shear compactor (GSC) at 100 gyrations, is truly enhanced; densities are indeed increased by 2.3% up to 3.8%. It is worthwhile to underline that this finding has been confirmed on site during an experimental roadwork near Blois (France) using the same asphalt formulas. Nonetheless, in this study, the use of 10% (continuously graded) RAP slightly decreases the compacting ability of the developed HPA (7.4% voids at 100 gyrations, instead of 5.9%), which indicates that non-negligible interference effects do occur between aggregate particles. Further work with 25% RAP will be set out in order to confirm or infirm this point.

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- Direct compressive strength (determined on the first set of cylindrical samples kept eight days in the air at 18°C), measured from the so-called Duriez test, is also truly enhanced (a difference from 1.7MPa up to 2.5MPa is obtained).

- Moisture resistance ("Duriez ratio") is far above the minimum specification value for typical Grave Bitume GB2. Nonetheless, the moisture resistance of the three tested HPAs is very slightly below that of reference GB2 material. This point needs further work since, in our opinion, there is Figure 15 in Annex 1 presents some photographs of GSC samples after 100 gyrations no clear reason for any decrease (even slight) in the moisture resistance of such well-interlocked and dense mixtures.

- Considering the resistance to rutting at 60°C, the added filler content of 10% brings about a clear increase in the permanent deformation susceptibility (rut depth of 10.2% at 10,000 cycles). Even with very good compactability, compressive strength, moisture resistance and even stiffness modulus, such HPA with 10% of added limestone filler is bound to develop high accumulated plastic deformations, *i.e.* rutting problems. However, the two HPAs with 5% added filler exhibit great resistance to rutting. This confirms the previous work of Perraton *et al.* (2007).

- Secant stiffness modulus measured at 15°C-0.02s is truly increased (in the range of 11% to 15%). Further work with different aggregate natures and fractions remains to be done so as to confirm this encouraging result. Indeed, the proposed procedure for optimizing the aggregate packing characteristics could be used in the framework of high-modulus mix design.

- Regarding the fatigue properties of HPAs, compared to those of the reference GB2 material, the two-point bending fatigue tests at 10°C-25Hz are still ongoing in the EIFFAGE Travaux Publics research centre. As such fatigue tests take some doing and for a lack of time, all the results will only be available in several weeks.

For illustration, Figure 15 in Annex 1 presents some pictures of GSC samples after 100 gyrations. Likewise, Figure 16 in Annex 2 presents some photographs of FWTT samples after 30,000 cycles.

## 9. Conclusions

Effective particle packing seeks to select proper sizes and proportions of small particle shaped materials to fill larger voids. These small particles in turn contain smaller voids that are filled with smaller particles, and so on. Such well-interlocked gap-graded mixtures have greater friction angles than the continuously dense-graded mixtures. Starting from such basic concepts associated with granular combinations, the aggregate packing methods first developed in the field of high-performance cement concretes were successfully transposed to the field of asphalt concretes and enabled the development and design of high-performance asphalts.

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A so-called "Grave Bitume" (continuously-graded) mixture, traditionally used for base courses in France, was studied, acting as a "reference" material. The first laboratory assessment of the optimal gap-graded mixtures which were designed in this study, were found to be very encouraging (the fatigue evaluation will be available in several weeks):

- the compacting ability, evaluated with gyratory shear compaction, is somewhat enhanced. In the framework of this work, with the same compaction energy, density values are more or less 3% higher,

- the resistance to moisture using the so-called Duriez test does not appear to be highly influenced by the optimal gradations used in this study. Further work is planned to confirm this point,

- the compressive strength values, determined at 18°C, increase by about 20%,

- the rutting resistance at 60°C-1Hz is not very influenced by the optimal gradations used on the condition that the filler addition does not exceed 5%. In the present study, a filler addition of 10% (close to the optimal value leading to a minimal porosity of the aggregate skeleton (after 20 gyrations at GSC), without any interference with the intermediate and coarse particles) makes the asphalt concretes more bound to rutting. Further work with a 7.5% filler addition will be performed in laboratory,

- the secant stiffness modulus measured at 15°C-0.02s is truly increased (in the range of 11% to 15%). Further work is planned to confirm this point.

From those first encouraging results, and as it was formerly observed in the field of high-performance cement concretes since the late 1970's, the use of "optimal" and dense gap-graded aggregate compositions brings about *i*) better workability and compacting ability, and *ii*) better compressive strength and stiffness modulus. The proposed principles are to be used to develop more specific guidelines for aggregate structure selection of high-performance asphalt concretes (HPAs).

In order to evaluate the application of HPAs on site, several experimental roadworks of with either 0 or 10 or 25% of RAP are from now on planned in France and Spain as well.

### Acknowledgements

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**Annex 1: GSC samples after 100 gyrations**

Compacting ability as measured from the gyratory shear compactor (GSC), following the requirements of the standard NF EN 12697-31. Three replicates were done for each test. Figure 15 presents some photographs of GSC samples after 100 gyrations. Image analysis is planned in order to assess the stone-to-stone contacts.

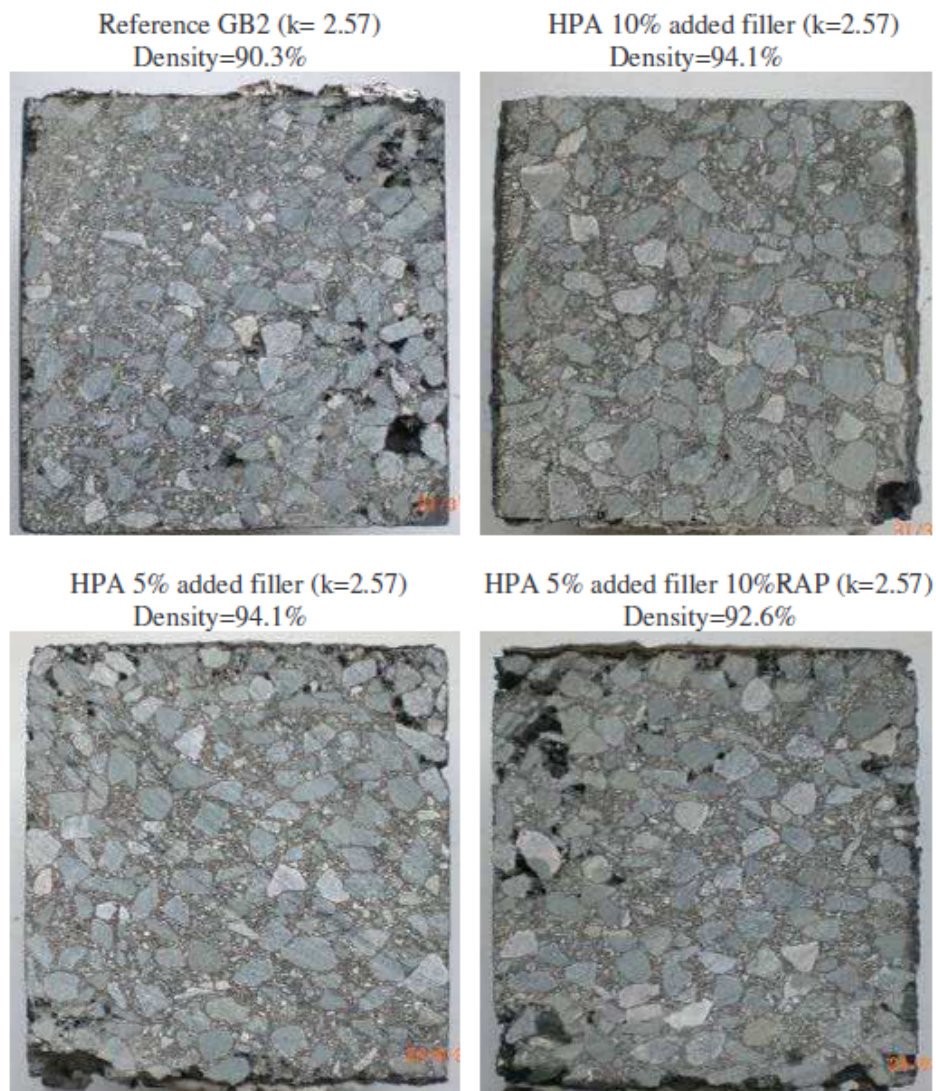


Figure 15. Slices of GSC samples after 100 gyrations.

**Annex 2: Rut Depth at 60°C and 30,000 cycles**

Rutting resistance was determined at 60°C with the French wheel tracking tester (FWTT) in accordance with NF EN 12697-22 (30,000 cycles, 1Hz). The test was run in duplicate.

Figure 16 presents some pictures of rutted samples after 30,000 cycles.

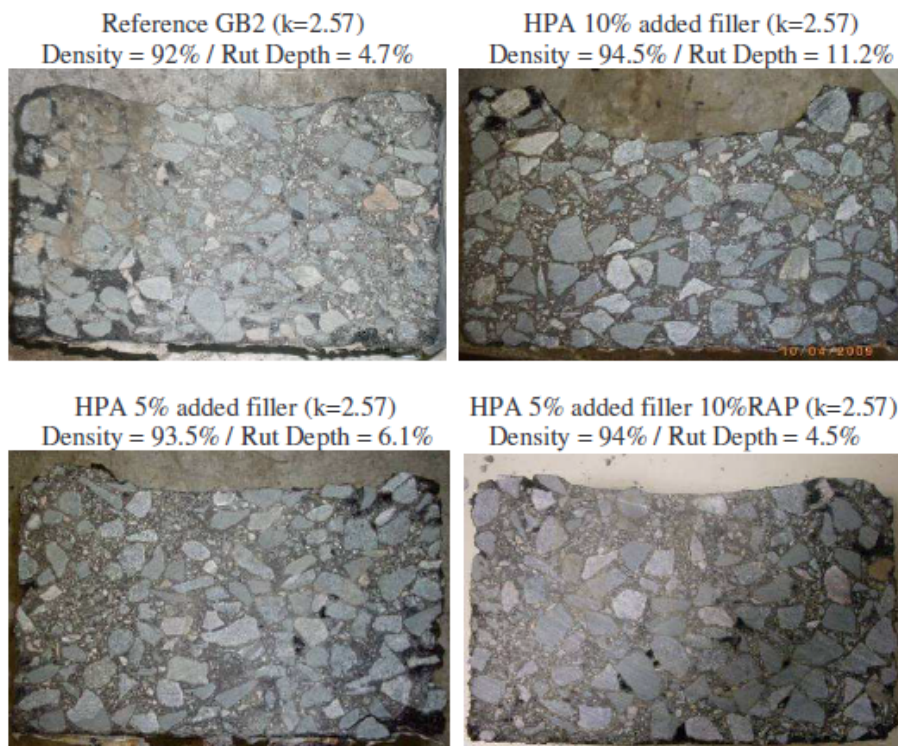


Figure 16. Slices of samples after wheel tracking test at 60°C and 30,000 cycles.



## **GB5 mix design: high-performance and cost-effective asphalt concretes by use of gap-graded curves and SBS modified bitumens**

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Aggregate packing concepts developed in the field of high-performance cement concretes, initially by Caquot (1937) then by contemporary researchers since the 1970s, were transposed to the field of asphalt concretes. These concepts, associated with the use of the gyratory compactor on aggregates only, enabled the development of a new laboratory design procedure of dense high-modulus asphalt concretes. These mixes are characterized by single or double gap-graded curves, great coarse aggregate interlock and no need for low penetration grade bitumens to fulfil the European EME2 specification requirements, in particular the 14,000 MPa stiffness modulus value at 15°C.

In addition, the use of polymer modified binders (PMBs), at a content of about 4% up to 4.5%, combined with such an optimized aggregate packing leads to the design of the so-called high-performance asphalt concretes (HPAs) characterized by great compactability, very high stiffness modulus and high fatigue resistance in a single formulation, allowing for reduced pavement thickness and increased longevity. Moreover, the proposed mix design and the 4–4.5% binder content makes PMBs use affordable in base courses. Laboratory assessment of such materials consisted in the evaluation of compactability, moisture resistance, rutting resistance at 60°C, complex stiffness modulus at 15°C and fatigue resistance at 10°C. Apart from these results, the paper also addresses the successful application of this new material on different job sites, located mainly in France. The proposed HPA material may be potentially considered as a relevant solution for sustainable long life pavements that do not deteriorate structurally, needing only timely surface maintenance.

**Keywords:** mix design; optimal aggregate packing; gap-graded asphalts; polymer modified binders (PMBs); high-performance asphalt mixes (HPAs)

### **1. Introduction**

Controlling the volumetrics and the compactability of asphalt concretes is the first step of any mix design procedure. Apart from the binder-related considerations, as the aggregate component represents 90 to 95% of the weight of asphalt mixtures, predicting and controlling the packing properties of aggregates is thus of prime importance. Broadly speaking, the aggregate packing characteristics are mainly influenced by the following five parameters (Caquot, 1937; Baron, 1982; de Larrard, 1988; de Larrard, Sedran, & Angot, 1994; de Larrard & Sedran, 1994, 2002; Corté & Di Benedetto, 2004):

- gradation (continuously-graded, gap-graded, etc.),
- shape (flat and elongated, cubical, round),

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- surface micro-texture (smooth, rough),
- type and amount of compaction effort (static pressure, impact or shearing),
- layer thickness (e.g. Cooper, Brown & Pooley, 1985).

This paper mainly focuses on the first parameter (gradation) by specifically optimizing the combination of fine and coarse fractions: the result is an interactive network of coarse particles in asphalt concrete, providing indirectly the strongest mix resistance (Roque, Huang, & Ruth, 1997; Kim, Guarin, Roque, & Birgisson, 2008) and in particular the highest mix modulus.

Apart from the previous gradation-related considerations, the ability of SBS polymers to reduce fatigue cracking and aging is well recognized (Baaj, di Benedetto, & Chaverot, 2005; Dreesen, Ponsardin, Planche, Dumont, & Pittet, 2011) but the high modulus required for perpetual pavement base structures usually calls for hard binders (and thus slightly higher binder content to preserve fatigue resistance) that have viscosity and compatibility issues with conventional SBS. Under those circumstances, at EIFFAGE Travaux Publics we set out to combine both optimal aggregate interlock (using aggregate packing methods initially developed in the field of high-performance cement concretes) and the use of SBS polymers, in order to obtain both very stiff and fatigue-resistant polymer modified base course material in a single formulation.

## 2. Theoretical background on aggregate packing

Many researchers developed empirical methods of relating air voids content in mineral aggregates to the gradation, or proposed 'ideal' gradations, which aim at maximum solid volume density. These theoretical curves are always continuously-graded curves and they generally have a parabolic form (Nijboer, 1948; Yoder, 1959). They all have a similar shape when placed on the same plot. The most prevalent theoretical 'ideal' gradation is based on the following empirical equation:

$$P = 100(d/D)^b \quad (1)$$

where:

- P*: percentage of aggregate, by weight, passing a particular sieve;
- d*: size of openings in the particular sieve, in millimetres;
- D*: maximum size of aggregate particles in the gradation, in millimetres;
- b*: coefficient. Nijboer (1948) and Yoder (1959) found that the maximum density of any continuously-graded compacted mix is obtained when *b* equals 0.45 or 0.5.

Despite equation (1), Lees (1970) emphasized that correct proportions for minimum void content must inevitably be affected by changes of aggregate shape from source to source and from size to size, by the level of compaction effort applied, by the presence of lubricating coatings, and by size and shape of the section in which the material is to be used. In addition, some more general concepts of aggregate packing were first developed by Caquot in 1937, and then by contemporary researchers since the 1970s, especially in the field of cement concrete (de Larrard, 1988; de Larrard, Sedran, & Angot, 1994; de Larrard & Sedran, 1994, 2002; Andersen & Johansen, 1993). A state of the art of basics has been recently presented by Perraton, Meunier, and Carter (2007) and Olard and Perraton (2010), transposing those concepts in the field of asphalt mix design. The following sub-sections are partially drawn from these papers.

### 2.1. Basic concepts associated with granular combinations

When filling a container with an aggregate or granular mix, just a fraction of the volume is occupied by the particles and the remainder is composed of interstices. It is worthwhile to remark that for an infinite medium, the void index ( $e = \text{volume of air voids}/\text{volume of solid particles}$ ) of an aggregate mix composed of one-dimensional particles remains, for all practical purposes, independent of particle size (Ben Aïm, 1967, 1970; Cumberland & Crawford, 1987).

When studying the porosity of mixes composed of two aggregates with differing yet one-dimensional individual sizes, Caquot first highlighted in 1937 the importance of two types of interparticle interaction on the void index: the so-called ‘wall effect’ and ‘interference effect’ – the latter is also called the ‘loosening effect’.

The ‘wall’ effect is tied to the interaction between particles and any type of wall (pipe, formwork, etc) placed in contact with the granular mass. The case of a uniform, two-aggregate mix can be considered in order to understand the concept. The two composing fractions only differ by their average particle dimension, i.e. one for the coarse aggregate particles and another for the fines. When adding a few coarse particles into an infinite volume of fines, the void index of the blend is reduced. Nevertheless, the coarse particles disturb locally (at the interface) the arrangement of fines whose porosity is increasing (i.e. an increasing void index). This local porosity increase is proportional to the particle surface area of the incorporated coarse aggregate (Caquot, 1937; Chanvillard, 1999). Figure 1 (left) and Figure 2 indicate the wall effect on the void index of a granular combination when raising the coarse fraction at the expense of fine aggregate within a binary mix.

By increasing the fraction of coarse particles within the fines, at a certain point a specific quantity of small particles winds up entrapped in the interstices delimited by the coarse particles. Therefore, outside of the wall effect, the fine aggregate void index increases due to interference: the arrangement of fine particles will depend not only on the surface areas of coarse particle walls (the wall effect), but also on the actual layout of these particles, i.e. the shape of their interstices.

The ‘interference’ (or ‘loosening’) notion can be illustrated by focusing on the effect induced by introducing a few fine particles into an infinite volume of coarse particles. As the amount of fines increases, at some point the coarse particles are forced apart by means of loosening, thus modifying their spatial configuration: interference occurs. Figures 1 (right) and 2 also display the interference effect on the void index of a binary granular combination.

If the average particle dimension of fines ( $d_{\text{FINE}}$ ) is small enough compared with the one of coarse particles ( $d_{\text{COARSE}}$ ) (e.g. when the  $d_{\text{FINE}}/d_{\text{COARSE}}$  ratio is lower than 0.2), the wall effect is linear and satisfies the superposition principle: the effect of two walls amounts to the sum of effects

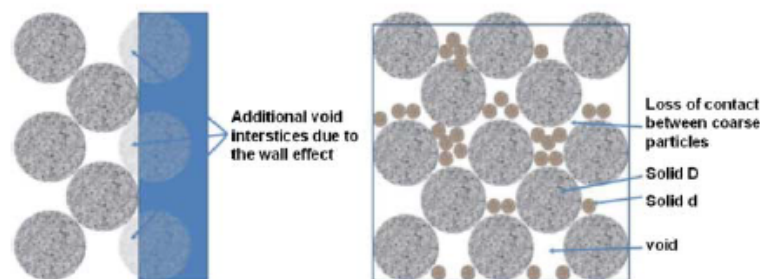


Figure 1. (Left) Schematic of the wall effect (case of the contact of fine particles at the surface of a coarse particle). (Right) Loosening effect (loss of stone-to-stone contact due to the all too high content of fine particles).



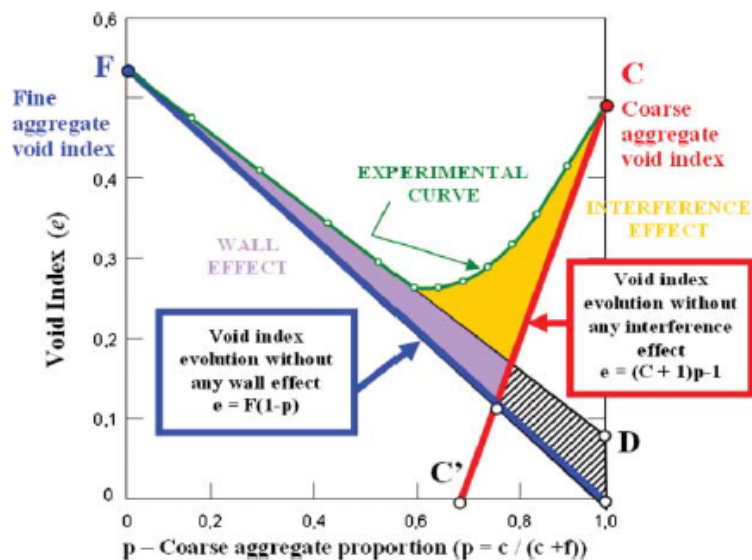


Figure 2. Schematic of the wall and interference effects (Powers, 1968).  $f$  and  $c$  are respectively the solid volumes of fine and coarse particles ( $f + c = 1$ );  $F$  (resp.  $C$ ) = void index of fine (resp. coarse) particles.

of each wall taken individually (Baron & Sauterey, 1982). In contrast, the interference/loosening effect is never linear and consequently difficult to frame simplistically (Baron & Sauterey, 1982).

### 2.2. Evolution in aggregate porosity versus average particle dimension

Furnas (1928), Powers (1968) and Oger (1987) showed the dependence of the shift in void index ( $e$ ) versus coarse aggregate portion in a binary combination on the ratio of average particle sizes. Figure 3 reveals that as the ratio of average fine aggregate dimension-to-average coarse aggregate dimension rises, interaction effects become more significant as well.

In order to reduce interactions of intermediate particles with the coarsest ones in the mix, it is crucial to limit both their size and amount and fill air voids by a higher fraction of fines instead. In addition, it appears that if the ratio between successive sizes in a gap gradation is chosen so as to give the most drastic reduction in voids, that reduction would be equal to, or possibly greater than, the most drastic reduction in voids for a continuous gradation.

### 2.3. Ideal case of a mix of an extremely fine aggregate with a coarse aggregate

For a situation in which one aggregate is very fine in comparison with the other ( $d_{\text{FINE}}/d_{\text{COARSE}} \sim 0.008$ ), Baron (1982) proposed describing the void index variation of a mix by means of three straight lines (Figure 4). Baron defined two thresholds,  $p_X$  and  $p_T$ , which indicate the critical concentrations that allow eliminating interference effects. Within a binary mix with coarse and fine particles, threshold  $p_X$  corresponds to the maximum coarse aggregate concentration that can be combined with fine aggregate without altering the fine aggregate arrangement, whereas threshold  $p_T$  is equal to the maximum fine aggregate concentration ( $1 - p_T$ ) for combination with coarse aggregate so as not to interfere with the coarse particle layout.

Depending on whether the granular mixture has a high ( $p < p_X$ ), medium ( $p_X < p < p_T$ ) or low ( $p > p_T$ ) content of fine aggregate, void index variation can be defined according to three

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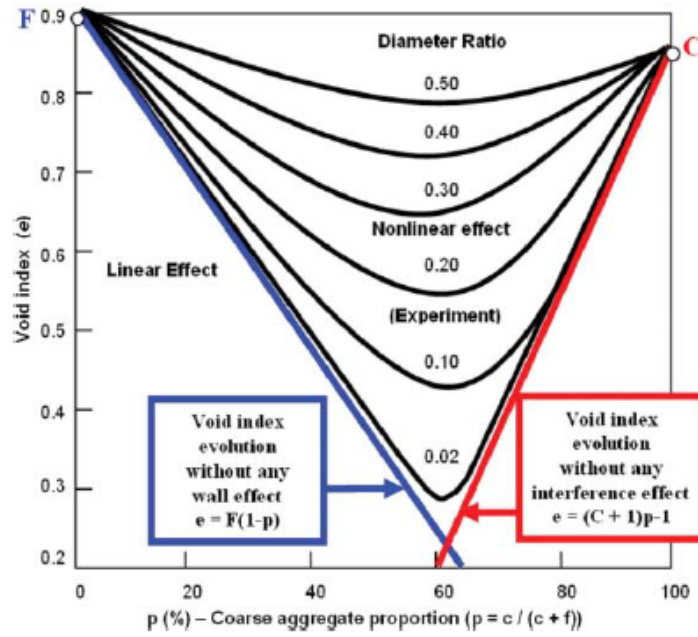


Figure 3. Effect of diameter ratio ( $d_{FINE}/d_{COARSE}$ ) on void index ( $e$ ) of binary granular blends (Furnas, 1928; Powers, 1968; Oger, 1987).

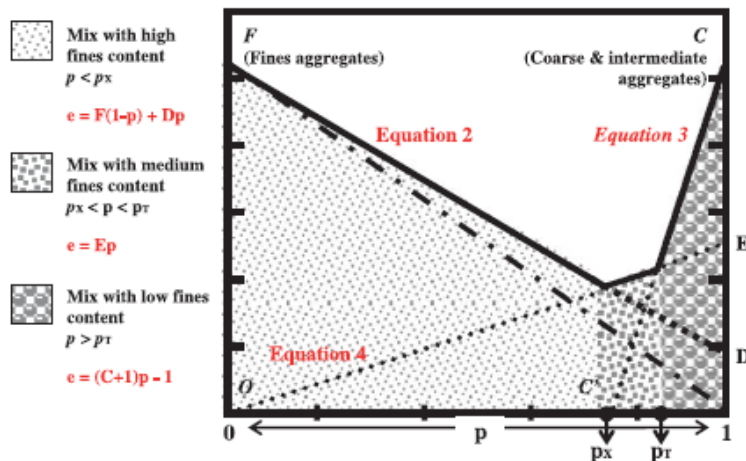


Figure 4. Void index variation ( $e$ ) in the case of a two-aggregate mixture, one of which is very fine compared to the other (according to Baron, 1982).

distinct laws, as shown in Figure 4:

- High content of fines in the mix,  $p < p_x$ :

$$\implies e = F(1 - p) + Dp \tag{2}$$

where  $F$  is the void index of fines and  $D$  is a parameter of the wall effect (Figure 4).

- Low content of fines in the mix,  $p > p_T$ :

$$\implies e = (C + 1)p - 1 \quad (3)$$

where  $C$  is the void index of coarse particles (Figure 4).

- Medium content of fines in the mix,  $p_X < p < p_T$ :

$$\implies e = Ep \quad (4)$$

where  $E$  is a coefficient without any simple physical significance.

#### 2.4. Transposition of Baron's approach to the design of stone-matrix asphalts with an optimal coarse aggregate packing according to Perraton et al. (2007)

By applying the concepts initially developed by Baron for the design of high-performance cement concretes, Perraton and his colleagues from the Ecole de Technologie Supérieure (University of Quebec) have recently illustrated that the laboratory performances of some SMA mixes may be substantially enhanced by means of a maximized coarse aggregate packing (Perraton et al., 2007).

The proposed mix design method was then applied to producing the so-called 'SMA-Cpack' (acronym for SMA with an optimal coarse aggregate packing) mixes with various NMPS (nominal maximum particle size) values using materials available in Montreal. Figure 5 shows a schematic diagram of the macrostructure in asphalt mix section cuts obtained using digital imaging. It may be noticed that the coarse particle content is high for each of the SMA-Cpack mixes produced.

First, the initial set of laboratory results suggests a potential binder saving of nearly 15% since the maximized coarse aggregate packing brings about a reduced void content in the granular skeleton. Second, based on Perraton's results, compactability and resistance to rutting of such mixes are remarkable.

### 3. Objective of the present study

The previous concepts associated with granular combinations and packing characteristics were used in the EIFFAGE Travaux Publics research centre in France. Baron's approach for optimal aggregate packing, developed in the field of high-performance cement concrete in 1982, was transposed and adapted to the field of asphalt concrete. The main goal was to evaluate the relevancy of such an approach in the asphalt field, especially with typical French aggregates. The underlying questions were: can we develop high-performance dense asphalts with multiple-gap grading mainly thanks to aggregate packing optimization? And, likewise, can we use more specific guidelines for aggregate structure selection?

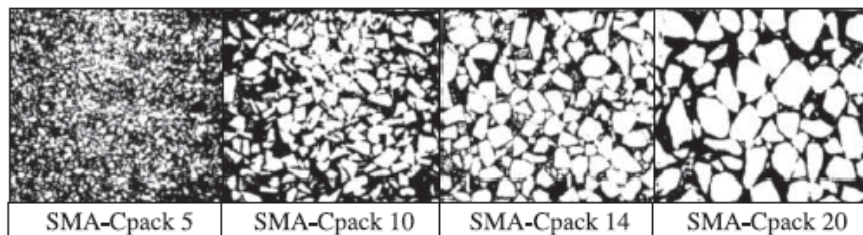


Figure 5. Illustration of the coarse aggregate proportion in the SMA-Cpack mixes (Perraton et al., 2007).

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After the publication of the first encouraging results obtained by Perraton et al. (2007) and after some technical exchanges with Perraton and his colleagues, EIFFAGE Travaux Publics has launched a large experimental program at its research centre, including the evaluation of moisture resistance, stiffness modulus and fatigue resistance as well. Laboratory tests consisted of the assessment of compactability using the French gyratory compactor (GC), moisture resistance using the Duriez test, rutting resistance using the French wheel tracking tester (FWTT) at 60°C, stiffness modulus at 15°C and finally fatigue resistance at 10°C.

#### 4. The proposed way of aggregate packing optimization

To make the simplest possible selection of an optimal granular combination with a maximized coarse fraction, it is herein proposed to separate the granular skeleton into three phases: fine, intermediate and coarse particles. The latter constitute the granular fraction that provides continuity in handling the force transfer within the mix macrostructure. This phenomenon is commonly referred to as coarse-on-coarse particle contact.

To enhance compactability, this method proposes monitoring the choice of granular combination in order to overcome the loosening effect by the intermediate fraction on the coarse fraction layout. In particular, the intermediate particles' dimension must be held below a certain critical value. With respect to Fumas' work (Figure 3), it is advised to limit the ratio of intermediate particle diameter ( $d_{INT}$ ) to coarse particle diameter ( $d_{COARSE}$ ) to 20% ( $d_{INT} \leq 0.20d_{COARSE}$ ) (see particle dimensions in Table 2). Likewise,  $d_{FINE} \leq 0.20d_{INT}$ .

Using the French GC on aggregates only – without any bitumen – the respective void index of coarse, intermediate and fine aggregate particles is first determined after 20 gyrations. Indeed, after 100 gyrations, attrition, segregation and abrasion may be observed (Figure 6).

Depending on the NMPS of the designed mix and thus on the number of used granular fractions ( $n$ ), the optimization sequence is performed during  $n - 1$  steps. For instance, in the case of a three-fraction mix with fine, intermediate and coarse aggregates, the optimization sequence is performed during two steps.

- Step 1: granular optimization of the intermediate aggregate–coarse aggregate blend (i.e. determination of the corresponding optimal ratio according to Baron's approach illustrated in Figure 4).
- Step 2: granular optimization of the blend between the previous optimized aggregate blend and the fine aggregates (same methodology, Figure 4).



Figure 6. Example of attrition, segregation and abrasion phenomena observed at 100 gyrations using the gyratory shear compactor.

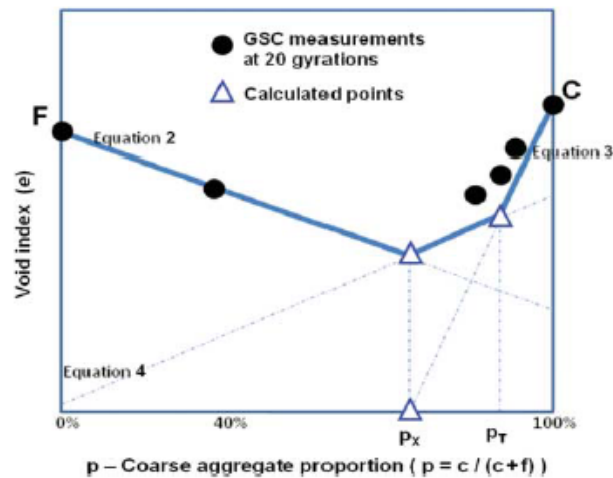


Figure 7. Experimental determination of the two thresholds  $p_x$  and  $p_T$ .

The only slight difference with Baron's original approach is that a sensitivity study is realized around threshold  $p_T$  (Figure 7). Two additional points are performed around  $p_T$  value ( $p_T \pm 3\%$ ). Moreover, for each step (i), equation (2) is defined by carrying out two GC tests: the first one with  $p = 0\%$ , the second one with  $p = 40\%$ . As illustrated in Figure 7, six GC tests are performed for each step (i). In the case of a three-fraction mix with fine, intermediate and coarse aggregate particles, granular optimization is performed with a series of 12 GC tests on this tertiary configuration.

Eventually, considering the effect of diameter ratio ( $d_{FINE}/d_{COARSE}$ ) on void index ( $e$ ) of binary granular blends (illustrated in Figure 3), it has been decided to introduce single or double gaps in granular gradations in order to obtain very dense aggregate packings with great coarse aggregate interlock (maximization of contact between coarse particles) and without any risk of the loosening effect (related to rutting issues).

## 5. Materials

Two pure paraffinic bitumens coming from the same crude and refinery were investigated: a straight run 35/50 and a semi-blown 35/45B ('B' stands for 'semi-blown') Pen grade bitumens. In addition, two PMBs made from these two pure bitumens with 2.5% SBS (proprietary crosslinking process) were also investigated. The analysis of the recovered aged binder of two RAP (reclaimed asphalt pavement) aggregates, both of them being studied hereafter, was also realized. Table 1 presents the results of conventional tests (Penetration at 25°C, Softening Point Ring and Ball and Fraass brittle point) initially performed on these different binders.

Because of the great number of aggregate combinations tested in the framework of this laboratory study, only two typical French aggregate natures are presented here: the diorite crushed aggregate fractions (0/2, 0/4 and 10/14 mm) coming from the 'La Noubleau' quarry and the limestone crushed aggregate fractions (0/2, 6/10 and 14/20 mm) coming from the 'Obourg' quarry. In both cases, added limestone filler coming from the 'St Hilaire' quarry in France was considered. Moreover, some RAP aggregates coming from either the 'Touraine Enrobés' asphalt plant (EIFFAGE TP Centre Ile-de-France) or the 'MEN' asphalt plant (EIFFAGE TP Nord) were used, respectively with the diorite and the limestone aggregate formulas. Table 2 and Figures 8 and 9 give the gradation curves of each granular fraction.

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Table 1. Conventional results on the studied binders.

Binder	Pen 25°C (mm/10) NFEN1426	Softening R&B (°C) NFEN1427	Fraass (°C) NFEN12593
35/50	38	53.5	-15
35/45B	37	62	-15
35/50 + crosslinked 2.5%SBS	38	62.2	-15
35/45B + crosslinked 2.5%SBS	33	71	-15
aged RAP binder 'Touraine Enrobés' plant	10	71.2	0
aged RAP binder 'MEN' plant	19	61.4	-5

Table 2. Passing percentage and average particle size for each granular fraction.

Sieve (mm)	Passing (%)								
	filler	Noubleau aggregates			RAP Tours	Obourg aggregates			RAP MEN
		0/2	0/4	10/14		0/2	6/10	14/20	
20								98	
16				100	100			56	
14				93	97			23	100
12.5				77	89		100	7.8	99
10				22	72		89	2.2	91
8				5	63		52	1.7	68
6.3			100	1.7	55		15	1.3	48
4		100	96	0.4	45	100	5	1.1	33
2		97	54	0.3	32	91	3	1	25
1		68	37	0.2	23	57	2	1	20
0.25	100	31	18	0.2	10	25	1	0.9	12
0.125	94	22	13	0.2	10	19	1	0.9	9
0.08	83	17	10	0.2	9	15	1	0.9	8
Average particle dimension, hereafter named $d_{50}$ Diameter (mm)	0.025	0.6	1.9	11.5	5	0.85	7.9	15.8	6.5

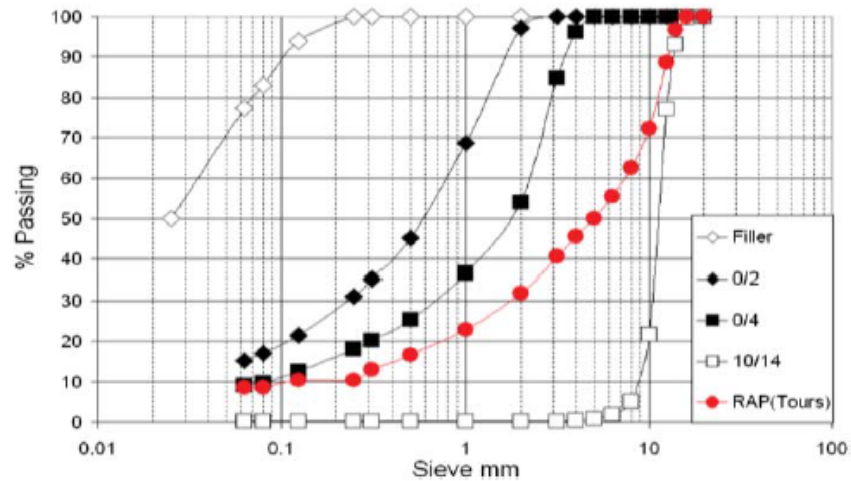


Figure 8. Gradation curves for each tested Noubleau granular fraction.

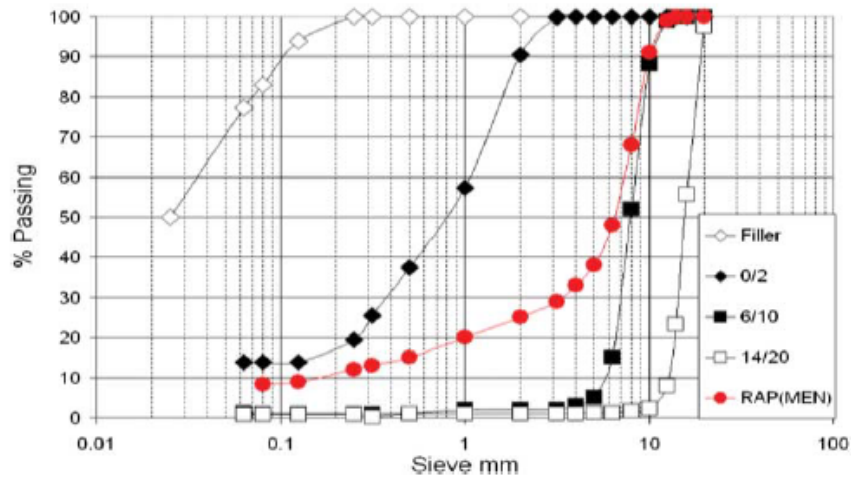


Figure 9. Gradation curves for each tested Obourg granular fraction.

Apart from the average dimension of added filler being determined by means of a Coulter particle size analyzer (NF ISO 13320-1, 2000), the average particle dimension of the other fractions was determined by sieve passing (cf. Table 2).

## 6. Description of the tests used for characterizing asphalt concretes

Many laboratory tests were conducted on asphalt concretes, including the following.

- Compactability, measured with the gyratory compactor (GC), following the requirements of NF EN 12697-31 standard (2005). This test gives a good idea of the job-site density values, according to course thickness. Conducted ahead of the other mechanical tests, this test is used to make a preliminary selection or screening of mixes, and for optimizing the asphalt mix composition (Harman, Bukowski, Moutier, Huber, & McGennis, 2002).
- Water resistance, measured from the so-called Duriez test (NF EN 12697-12, 2002) which consists of a direct compression test on two sets of six cylindrical samples, one set of six samples tested after conditioning in water. If the ratio of average compression strengths after and before conditioning is above a certain value, the material is deemed to be acceptable. This ratio is the French counterpart of the TSR (Tensile Strength Ratio) value with Marshall samples.
- Resistance to rutting at 60°C, determined with the FWTT following NF EN 12697-22 standard (rectangular samples subjected to repeated passes at 1 Hz of a wheel fitted with a tyre, inducing permanent deformation, 2004).
- Complex stiffness modulus measured at 15°C–10 Hz, in accordance with the NF EN 12697-26 requirements (strain-controlled test on cylindrical specimens, 2004). Complex modulus test was also performed for some high-performance asphalt mixes (HPAs) from –30°C to 45°C with a frequency ranging from 0.01 to 10 Hz.
- Complex stiffness modulus and fatigue resistance at 10°C–25 Hz, following the NF EN 12697-24 standard (controlled-strain test on trapezoidal specimens with unconfined conditions, see Figure 10, 2005). The fatigue criterion that is used in this paper is the classical one, referenced as  $Nf_{50}$ . It corresponds to the number of cycles for which either the complex

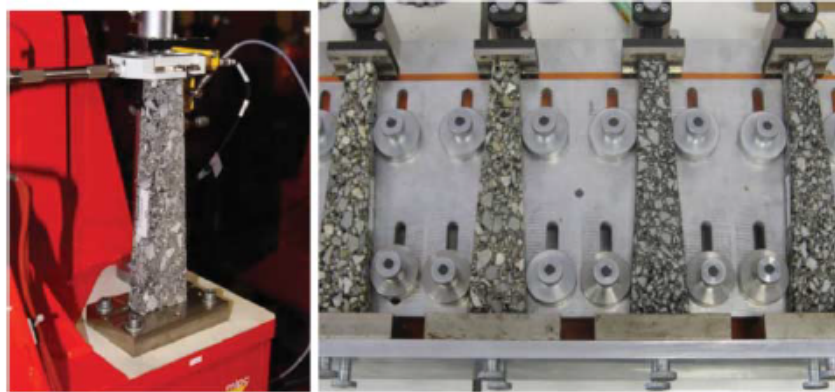
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Figure 10. Complex modulus test and fatigue test configuration on trapezoidal specimens (NF EN 12697-24, 2005).

modulus decreases to 50% of its initial value, or a sudden failure occurs (before 50% decrease of modulus). The value of the strain amplitude leading to failure at one million cycles is hereafter called  $\varepsilon_6$ . This parameter is used in the French method SETRA-LCPC (1997).

## 7. Aggregate packing optimization results

The use of a single-gap or even a double-gap gradation may be very helpful in obtaining very dense asphalt concretes with an outstanding coarse aggregate packing (there is far less interaction between intermediate and coarse particles, cf., for example, Figure 3). Thus, such single-gap and double-gap gradations were respectively investigated with Noubleau and Obourg aggregates.

### 7.1. Optimization of single-gap-graded granular curve with Noubleau aggregates

Figure 11 illustrates the three-step iterative aggregate packing optimization of a quaternary 10/14-0/4-0/2-filler aggregate combination (4/10 mm gap), by using the gyratory compactor (GC) on aggregates only – at ambient temperature and without any bitumen – as previously detailed.

The optimal 10/14-0/4-0/2-filler quaternary blend is the following:

- 10/14 content: 55.3% ( $=0.865 \times 0.64$ )
- 0/4 content: 13.9% ( $=0.865 \times 0.16$ )
- 0/2 content: 17.3% ( $=0.865 \times 0.2$ )
- added filler content: 13.5% ( $=1-0.865$ )

At the very last stage of such optimization, when determining the optimal content of added filler, the three theoretical straight lines described by Baron are clearly validated (Figure 4 and equations (2) to (4) in the particular case of a binary blend of very fine particles and coarse particles). Nevertheless, even if the addition of 13.5% of filler is quite convenient and relevant in the field of cement concretes, where final air voids are close to zero and production is realized on discontinuous batch plants, the added filler content was arbitrarily fixed at 5% in order to avoid overfilled asphalt concretes in the end and as the production may be realized on continuous drum mix plants. Table 3 and Figure 12 show gradation curves of each tested asphalt concrete. Figure 12



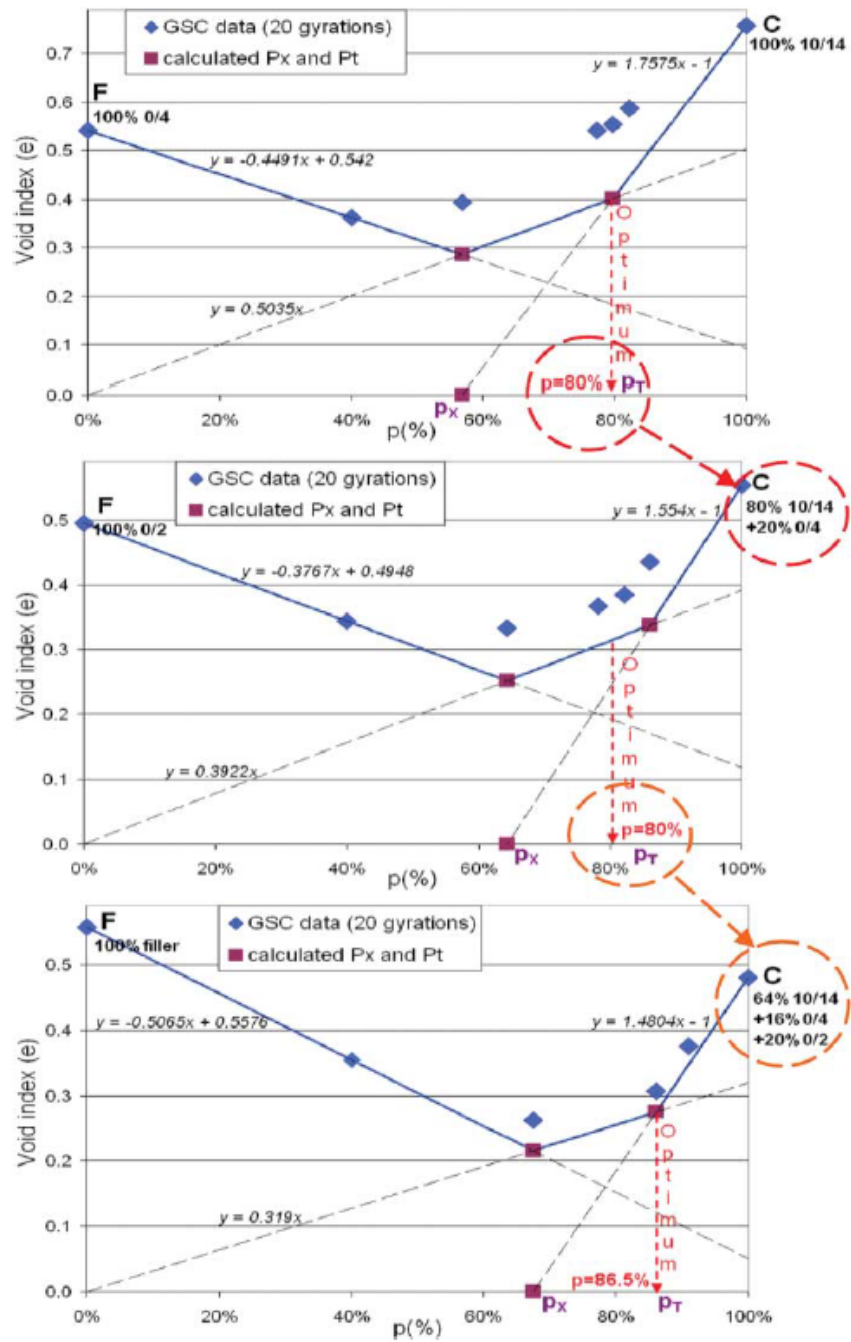


Figure 11. Three-step iterative optimization of the 10/14-0/4-0/2-filler blend.

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Table 3. Passing percentage for each tested asphalt mix made from the Noubleau aggregates.

Sieve (mm)	Passing (%)		
	Reference Grave Bitume GB2 0/14	HPA 0/14 with 5% added filler (4/10 gap-graded)	HPA 0/14 with 5% added filler (4/10 gap-graded) + 10% RAP
20	100	100	100
16	96	96	96
14	94	92	93
12.5	88	83	84
10	67	51	54
8	57	42	44
6.3	50	40	42
4	42	38	39
2	34	32	32
1	25	24	24
0.25	13	14	14
0.125	9	11	11
0.08	8	9	9

Table 4. Passing percentage for each tested asphalt mix made from the Obourg aggregates.

Sieve (mm)	Passing (%)		
	Reference Grave Bitume GB2 0/20	HPA 0/20 with 5% added filler (2/6 & 10/14 gap-graded)	HPA 0/20 with 5% added filler (2/6 & 10/14 gap-graded) + 10% RAP
20	100	100	100
16	91	83	83
14	87	70	70
12.5	78	65	66
10	70	59	60
8	64	49	49
6.3	57	38	39
4	41	34	35
2	27	31	31
1	18	22	22
0.25	10	12	12
0.125	8	10	10
0.08	7	8	8

shows in particular that the passing of the different HPA aggregate gradations at the 2-mm sieve is above the SMA window (NF EN 13108-5, 2006), which is indirectly due to the 4/10 mm gap gradation.

A typical gradation curve of a 'Grave Bitume 2' material (GB2) is also presented, for comparison.

### 7.2. Optimization of double-gap-graded granular curve with Obourg aggregates

The proposed methodology was applied to optimize the double-gap-graded granular curve with the so-called Obourg aggregates. To the extent that four granular fractions were investigated (14/20 mm, 6/10 mm, 0/2 mm and filler), optimization of aggregate packing was realized in three steps:

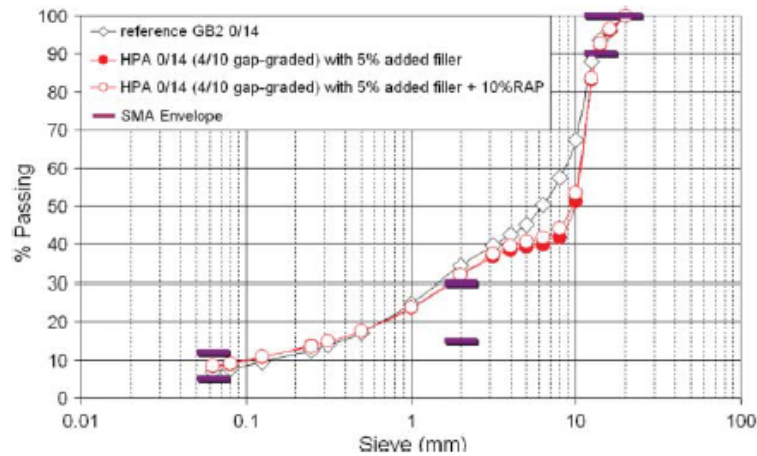


Figure 12. Gradation curves for the optimal 10/14-0/4-0/2-filler quaternary Noubleau aggregate blends versus typical gradation curve of the reference 'GB2'.

- Step 1: optimization of the binary blend composed by 14/20 and 6/10 ( $p = 60\%$  (i.e. 60% 14/20 and 40% 6/10), cf. Figure 13). As  $d_{50}(6/10)/d_{50}(14/20) = 0.50$ , a true interference occurs in the case of the 14/20-6/10 binary aggregate composition (see Figure 3). The optimal content of coarse aggregate 14/20 may be arbitrarily fixed at 60% (leading to the densest binary blend).
- Step 2: the optimal blend obtained in Step 1 (60% 14/20-40% 6/10) is considered as the 'coarse fraction', whereas the 0/2 fraction is considered as the 'fine fraction'. Optimal coarse ratio is  $p_T = 70\%$ . It is noteworthy that  $d_{50}(0/2)/d_{50}(6/10) \approx 0.11$ , leading to negligible interference between 0/2 and 6/10 particles (see Figure 3).
- Step 3: the optimal blend obtained in Step 2 (42% 14/20-28% 6/10-30% 0/2) is considered as the 'coarse fraction', whereas the filler is then considered as the 'fine fraction'. Optimal coarse ratio is  $p_T = 95\%$ . Note that  $d_{50}(\text{filler})/d_{50}(0/2) \approx 0.03$ , hence next to no interference between filler and 0/2 particles is observed (see Figure 3). In addition, the three straight lines from Baron's approach cannot be obtained (Figure 4 and equations (2) to (4)).

The optimal 14/20-6/10-0/2-filler quaternary blend is the following:

- 14/20 content: 40% ( $= 0.95 \times 0.42$ )
- 6/10 content: 26.5% ( $= 0.95 \times 0.28$ )
- 0/2 content: 28.5% ( $= 0.95 \times 0.30$ )
- added filler content: 5% ( $= 1 - 0.95$ )

Figure 14 shows in particular that the passing of the different HPA aggregate gradations at the 2-mm sieve is above the SMA window (NF EN 13108-5, 2006), which is indirectly due to the 2/6- and 10/14-gap gradation.

## 8. Performance characterization and related discussion

In the framework of our study with Noubleau or Obourg mixtures, a 4% binder content by weight of aggregate (10% vol.) was used. Tables 5 and 6 respectively present the performances of HPAs

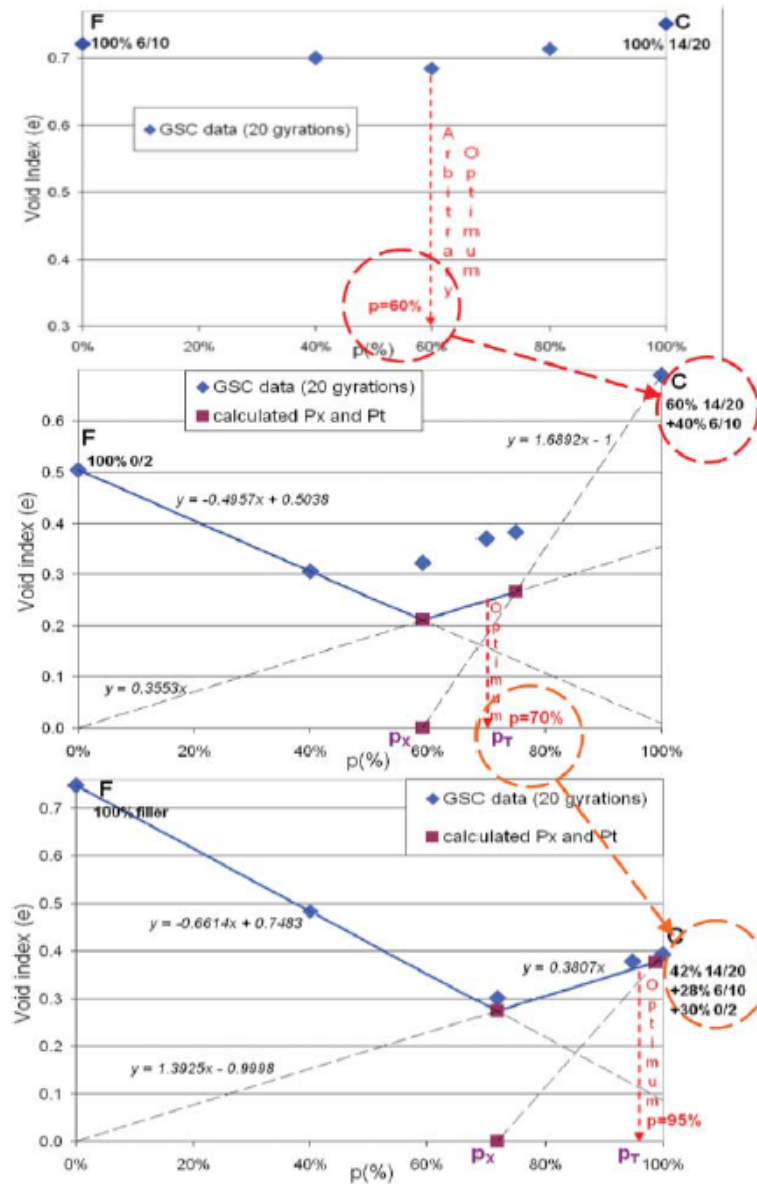
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Figure 13. Three-step iterative optimization of the 14/20-6/10-0/2-filler blend.

made from Noubleau and Obourg aggregates, compared with the reference continuously-graded 'Grave Bitume 2' (GB2) material normally used in France.

French specifications for a continuously-graded GB2 and 'Enrobé à Module Elevé 2' (EME2) are given as well. The reference EME2 material consists of a continuously-graded curve and relies on the use of hard binders (Penetration at 25°C < 30 dmm) at a content of 5.5–6.0% by weight of aggregates to preserve fatigue resistance. EME2 is traditionally used as base course material in Perpetual Pavement design in Europe and, in particular, in France.

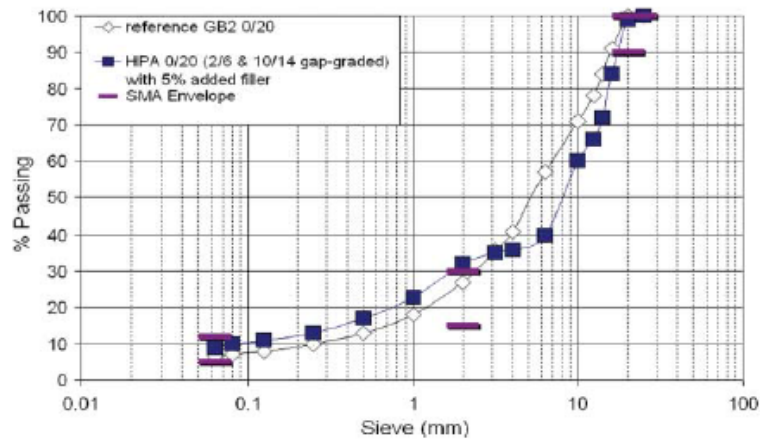


Figure 14. Gradation curves for the optimal 14/20-6/10-0/2-filler quaternary Obourg aggregate blends versus typical gradation curve of the reference 'GB2'.

Table 5. Performances of HPA's and reference GB2 material. Noubleau aggregates with a 4% binder content by wt of aggregate (%vol. = 10%).

Formula	GC 100 gyrations			Duriez Test		Rut Depth (mm) $3 \cdot 10^4$ cycles	E (MPa) 15C-10 Hz	$\epsilon_6$ ( $10^{-6}$ ) 10C-25 Hz
	Binder nature	%RAP	%Air	VMA	R (MPa)			
Specifications for 'Grave Bitume 2' (GB2)		<10%	-	-	>70%	<10	>9000	>80
Specifications for 'Enrobé à Module Elevé 2' (EME2)		<6%	-	-	>70%	<7.5	>14000	>130
GB2 35/50	0	9.7	19.7	10.1	93	4.1	13,300 at 5.1%air	86
	0	5.9	15.9	11.8	83	5.1	16,500 at 2.7%air	89
	10	7.2	17.2	12.1	91	4.1	16,600 at 2.7%air	90
HPA 35/50 + 2.5%SBS	0	5.7	15.7	12.3	93	2.4	15,600 at 3.2%air	115
35/45B		5.8	15.8	12.7	92	2.5	13,100 at 2.9%air	115
35/45B + 2.5%SBS		5.7	15.7	13.1	91	3.0	13,700 at 2.5%air	130

### 8.1. Compactability evaluated from the gyratory compactor

Compactability, measured from the gyratory compactor (GC) at 100 gyrations, is significantly improved:

- regarding HPAs made from Noubleau aggregates (with single-gap-graded curve), densities are increased by 2.3% up to 3.8%. Nonetheless, the use of 10% (continuously graded) RAP slightly decreases the compactability of the HPA (7.2% air voids at 100 gyrations, instead of 5.9%), which indicates that non-negligible interference effects do occur between aggregate particles. Further work with 25% RAP will be set out to confirm this point;
- regarding the HPAs made from Obourg aggregates (with twice-gap-graded curve), densities are indeed increased by about 5%. The use of such optimized double-gap-graded granular curves appears as most promising.

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Table 6. Performances of HPAs and reference GB2 material. Obourg aggregates with a 4% binder content by wt of aggregate (%vol. = 10%).

Formula	GC 100 gyrations		Duriez Test		Rut Depth (mm) $3 \cdot 10^4$ cycles	E 15C-0.02 s (MPa)	$\epsilon_6$ 10C-25 Hz ( $10^{-6}$ )
	Binder nature	%Air	VMA	R (MPa)			
Specifications for 'Grave Bitume 2' (GB2)	<10%	–	–	>70%	<10	>9000	>80
Specifications for 'Enrobé à Module Elevé 2' (EME2)	<6%	–	–	>70%	<7.5	>14000	>130
GB2 35/50	8.7	18.7	11.8	83	5.1	13,900 at 7.0%air	89
35/45B	8.9	18.9	11.3	84	3.3	10,800 at 7.2%air	109
HPA 35/45B	3.8	13.8	12.7	85	3.4	13,000 at 3.6%air	114
35/45B + 2.5%SBS	3.8	13.8	12.5	83	2.1	12,800 at 4.3%air	135

It is worthwhile pointing out that such an excellent compactability of HPAs made from either Noubleau or Obourg aggregates has been confirmed on site during experimental roadworks near Blois, Tours and Arras (France) using the same asphalt formulas.

Eventually, even with a somewhat low value of binder content (%vol. = 10%), the proposed single- or double-gap graded curves allow one to obtain very encouraging VMA results (from 13.8% to 15.8%).

### 8.2. Compressive strength and moisture resistance assessed from the Duriez test

Direct compressive strength (determined on the first set of cylindrical samples kept 8 days in the air at 18°C), measured from the so-called Duriez test, is also improved (an increase from 1.2 MPa to 2.5 MPa is obtained).

In addition, moisture resistance ('Duriez ratio'), which is far above the minimum specification value for typical Grave Bitume GB2, does not seem to be influenced by the innovative gradations and PMBs used.

### 8.3. Rutting resistance assessed from the French wheel tracking test

The HPAs exhibit great resistance to rutting (low values of rut depth at 60°C after 30,000 cycles, cf. Tables 5 and 6) for two main reasons: the first reason is due to the well-interlocked and dense mixtures obtained from the optimization of aggregate packing, whereas the second reason lies on the fact that semi-blown and polymer modified bitumens give high resistance to rutting at high temperatures, in particular at 60°C.

### 8.4. Complex Modulus

For a fixed bitumen nature and content, the complex stiffness modulus of such well-interlocked and dense mixtures, measured at 15°C–10 Hz, is significantly increased (in the range of 20% to 26%). Further work with different aggregate natures and fractions remains to be done so as to confirm this encouraging result. Indeed, the proposed procedure to optimize the aggregate packing characteristics could be used in the framework of high-modulus mix design with slightly softer

grades than usual (Penetrability at 25° > 30 in contrast with the reference European EME2 mix design), thus enhancing both the reclaiming ability and fatigue resistance of asphalts concretes.

In addition, in order to assess the influence of the proposed gap-gradation on both glassy and static moduli, complex stiffness modulus tests were also performed for the GB2 reference material and the proposed gap-graded HPA with Noubleau aggregate, with the same binder (35/50) at the same amount (4% binder content by weight of aggregate), from -30°C to 45°C with a frequency ranging from 0.01 Hz to 10 Hz (see Figure 15).

Regarding the glassy modulus (asymptotic value of complex modulus in the low-temperature and high-frequency domain), an increase by 20% was obtained, whereas for the static modulus (asymptotic value in the high-temperature and low-frequency domain), a threefold increase was obtained. These first results could make such HPAs interesting for heavy duty pavements, heavy duty bridge deck pavements, and so on.

8.5. Fatigue resistance

The very first results of HPA fatigue resistance tests appear comparable with those of the reference GB2 material, for a fixed bitumen nature and content. Fatigue resistance is indeed hardly influenced

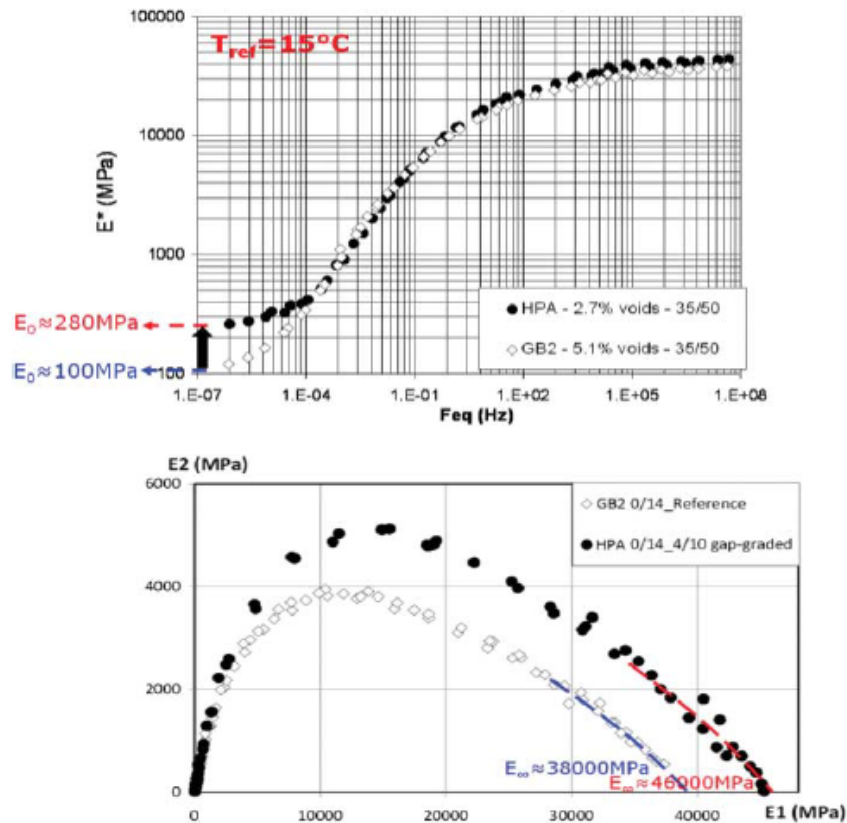


Figure 15. Complex modulus of the gap-graded HPA versus that of the reference GB2 (Noubleau aggregate & 4% of 35/50 binder by wt of agg.): Master Curves at 15°C and Cole-Cole diagram.

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Figure 16. Paving GB50/14 mm with Noubleau aggregates and 35/45B bitumen in the Tours area. In-place density: 96.4%; stiffness modulus measured in indirect tension configuration  $E^*(15^\circ\text{C}-10\text{ Hz})$ : 14,100 MPa.

by the developed aggregate packing optimization. On the contrary, binder nature highly influences fatigue resistance:

- with regards to the HPA with Noubleau aggregate (with single-gap-graded curve), the polymer modification (2.5% cross-linked SBS) of the 35/50 neat bitumen leads to an increase of the  $\varepsilon_6$  value by 21 microstrains;
- regarding the reference 'GB' made from Obourg aggregates (with continuous gradation), moving from the neat 35/50 bitumen to the blown 35/45B bitumen brings about an increase of the  $\varepsilon_6$  value by 20 microstrains;
- regarding the HPA with Obourg aggregates (double-gap-graded curve), the polymer modification (2.5% cross-linked SBS) of the blown 35/45B bitumen increases the value of  $\varepsilon_6$  by 21 microstrains, leading to an  $\varepsilon_6$  value of 135 microstrains, which is an incredibly high value for an asphalt concrete with only a 4% binder content in France.

An attempt was done to adjust the GB2 blends in order to lower VMA values close to those of the optimized blends to see the effect on the stiffness and fatigue performances. Yet it was impossible to get the 2.7% air voids target.

### 9. Economic and environmental outlook

An alternative to the traditional high-modulus and high-binder content EME2 (for which binder content is about 5.5% to 6%) may be proposed for long-lasting and cost-effective asphalt mixes. Considering the very encouraging results presented in Tables 5 and 6 (with only 4% of bitumen by weight of the aggregate), at EIFFAGE Travaux Publics we set out to combine both optimal aggregate interlock and the use of semi-blown and/or SBS modified bitumens, so as to obtain both very stiff and fatigue resistant base/binder course material in a single formulation.

This has been done with many aggregate natures (mainly from France) by using either single-gap or double-gap graded curves and a tremendous number of polymer modified and semi-blown bitumens. Obtained performances are close to the specifications required for EME2 (stiffness modulus of 14,000 MPa at 15°C and a fatigue resistance of 130 microstrains at 10°C) with a



significantly lower bitumen content (in the range of 3.9% to 4.9% by weight of aggregate). Such innovative mix design, which is referred to as GB5, has been patented.

The two following sub-sections present a case study with comparative pavement design and environmental impact.

### 9.1. Hypothesis for pavement design and material cost

The French method for pavement design has been used. Calculations presented hereafter rely on the use of the so-called ALIZE software. The adjustment factor, named  $k_c$ , that is used to determine the strain value  $\varepsilon_{t,ad}$  considered acceptable at the bottom of the GB5 base course, is considered as equal to 1.3 (typical value for a French Grave Bitume (GB) base layer). Furthermore, the  $k_c = 1$  hypothesis is made when considering conventional high modulus asphalt concretes (referred to as EME2), which use very low Pen grade bitumens (Penetrability at 25°C in the range 10–30 dmm). Broadly speaking, this  $k_c$  coefficient adjusts the results of the computational model in line with the behaviour observed on actual pavements of the same type. For more details, the value of this coefficient, for bituminous materials, is detailed in the French design manual for pavements structures (SÉTRA-LCPC, 1997).

In order to compare the costs of road structures with traditional EME2 or innovative GB5 base courses, the following orders of magnitude were taken into account for material costs:

- cost of 10/20 hard bitumen  $\approx$  35/50 pen grade bitumen + 40–60 €/t,
- cost of 35/45B  $\approx$  35/50 pen grade bitumen + 40–60 €/t,
- cost of 35/50 + 2.5%SBS  $\approx$  35/50 pen grade bitumen + 150–170 €/t,
- cost of 35/45B + 2.5%SBS  $\approx$  35/50 pen grade bitumen + 200–220 €/t.

The apparent specific gravities (ton/m<sup>3</sup>) are:

- $\rho(\text{BBM})$ : 2,42 t/m<sup>3</sup>,
- $\rho(\text{GB5})$ : 2,47 t/m<sup>3</sup>,
- $\rho(\text{EME2})$ : 2,49 t/m<sup>3</sup>.

A 'Béton Bitumineux Mince' (BBM) is used as a 4 cm-thick overlay in France.

### 9.2. Comparative pavement design, corresponding costs and environmental impacts

Table 7 presents a comparative pavement design using the ALIZE software and considering a 'TC620' traffic category (cumulative traffic over 20 years in the range between 6.5 and 17.5 million 13-ton axles), a 4 cm-BBM overlay and a 'PF3' pavement formation class (120 MPa). Materials are the same ones listed in Table 5, together with their respective performance results.

Table 7 shows that innovative GB5 mixes do have very positive environmental and economical impacts when considering the reduced base layer thickness and the reduced quantities of binder and aggregate required per square metre.

Insofar as greenhouse gas emissions (GGEs) are concerned, carbon dioxide (CO<sub>2</sub>) quantities (main GGE generated during roadwork) associated with aggregates, and pure bitumen and polymer modified binder are respectively equal to 10, 285 and 310 kgCO<sub>2</sub>/t. Therefore, the proposed high-performance GB5 base layers may lead to a reduction in CO<sub>2</sub> emissions by almost 30% in comparison with traditional EME2-based pavement design (see Table 7).

However surprising it may seem, the proposed aggregate packing optimization makes the SBS affordable in base layer materials (its use was limited until now to surface courses for economic reasons).

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Table 7. Comparative pavement design with Noubleau materials (from Table 5), corresponding costs &amp; related environmental impacts (per square meter).

Thick bituminous pavement structures. 'TC620' traffic category. 4 cm-BBM Overlay. 'PF3' pavement formation class.				
	EME2 Solution Binder content = 5, 7%	Innovative GB5 <sup>®</sup> Solutions Binder content = 3, 9%		
Binder	Hard 10/20	35/45B	35/50 + 2.5%SBS	35/45B + 2.5%SBS
Overlay	4 cm BBM	4 cm BBM	4 cm BBM	4 cm BBM
Base course	16 cm EME2	14 cm GB5	12 cm GB5	10 cm GB5
Difference in base layer thickness	Reference	-2 cm (-10%)	-4 cm (-20%)	-6 cm (-10%)
Difference in aggregate quantity		-10%	-20%	-30%
Difference in bitumen quantity		-28%	-39%	-48%
Difference in materials cost/m <sup>2</sup>		-23%	-27%	-38%
Difference in kg CO <sub>2</sub> eq./m <sup>2</sup>		-24%	-17%	-28%

#### 10. In-situ validation of the proposed mix design in the Year 2010

Ten full-scale suitability tests were first carried out on several EIFFAGE asphalt plants in 2010 before generalizing the technique on each of the 180 EIFFAGE sites. These preliminary field trials allowed validating the technical choices with either single-gap or double-gap gradations initially studied in the laboratory. In particular, the great compactability of the proposed mixes was confirmed so far.

#### 11. Development and large-scale roadworks of 2011

In 2011, the use of the proposed GB5 mix design was generalized on most French EIFFAGE sites. Several aggregate natures were studied. Four main nominal maximum particle sizes (NMPS) were used: 11 mm, 14 mm, 16 mm and 20 mm. Both single gap-graded curves and double gap-graded granular curves were successfully investigated. Semi-blown 20/30, 35/50, 50/70 and 70/100 Pen grade binders were used. SBS modification was also carried out (referred to as Biprene and Orthoprene, with a proprietary cross-linking procedure) for most of our full-scale suitability tests so far. In PMB's compositions, SBS content was in the range of 2.5% to 7%. The overall binder content (virgin + aged binders) was generally in the range of 4.0% to 4.5%.

The so-called GB5 mixes were applied by the paver and very easily compacted by double-roll vibrating compactors (Figures 17 and 18). There was no need for pneumatic tyre rollers, final density being in the range between 2% and 6%. Layer thickness was between 6 and 16 cm thanks to gap gradations used.

140,000 tons of GB5 mixes have been paved so far, corresponding to a saving of about 1400 tons of bitumen in comparison with the reference EME2 (high-modulus high-binder content mix) usually used as base course material in Perpetual Pavement design in Europe. On top of it, almost 4000 tons of PMBs have been produced and used so far in base courses, whereas their use was limited until now to surface courses for economic reasons.

After several months (trials in 2010–2011), these different sites were revisited in order to assess the condition of the pavement and/or to take cores to assess density and complex or secant



Figure 17. GB5 0/14 mm with Budillon aggregate and SBS modified binder Biprène 41 (proprietary composition) and 15% RAP near Chambéry. In-place density: 97%; modulus  $E^*$  (15°C–10 Hz): 15,900 MPa.

modulus in IDT (indirect tension) mode. This follow-up is very encouraging and confirms the great performances initially obtained in the laboratory.

One initial concern was to check the validity and effectiveness of the proposed approach with mixes containing high RAP contents. In 2011, some GB5 mixes were successfully designed and manufactured with 40% RAP: RAP aggregate size was 0/10 mm and the added crushed aggregate size was 14/20 mm, in order to get a single 10/14 mm gap in the 0/20 mm gradation. Further work is planned within the next few years on this issue, by using specially fractionated recycled materials. By fractionating RAP and combining high RAP contents with unaged SBS-modified binder, the GB5 mix design could produce a longer life pavement with a very interesting economic outlook.

Last but not least, in the framework of the so-called Road Innovation Charter procedure of SETRA (French acronym which stands for 'Service d'Etudes Techniques des Routes et Autoroutes'), in France, the innovative GB5 project was awarded in 2010. In 2011, a Road Innovation Charter was signed with SETRA and several General Councils and companies in charge with toll highways. From then on, several GB5 projects were undertaken in different France's climatic zones under very high traffic. A five-year follow-up by SETRA is planned for validation of this new technique.

## 12. Conclusions

Effective particle packing seeks to select proper sizes and proportions of small particle-shaped materials to fill larger voids. These small particles in turn contain smaller voids that are filled with smaller particles, and so on. Such well-interlocked gap-graded mixtures have greater friction angles than the continuously dense-graded mixtures. Starting from such basic concepts associated with granular combinations, aggregate packing methods first developed in the field of high-performance cement concretes were successfully transposed and adapted to the field of asphalt concretes and enabled the development and design of high-performance asphalts. The proposed mix design method is from now on referred to as the GB5 mix design method in France.

A so-called 'Grave Bitume GB2' (continuously-graded) mixture, traditionally used for base courses in France, was studied, acting as a 'reference' material. The first laboratory assessments

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Figure 18. 12,000-ton GB5 0/14 roadwork on the French A43 toll highway (AREA network, Chignin-Francin section) in the alpine area. Budillon aggregate, 15% RAP (reclaimed asphalt pavement) and PMB's Biprene 41 & Orthoprene were used.

of the optimal single or double gap-graded mixtures designed in this study were found to be very encouraging:

- compactability, evaluated through gyratory shear compaction, is remarkably improved. In the framework of this work, for the same compaction energy, density values are approximately 4% higher;

- moisture susceptibility, assessed using the so-called Duriez test, does not appear to be influenced by the proposed optimal gradations. Further work is planned to confirm this point;
- compressive strength value at 18°C is increased by about 15%;
- rutting resistance at 60°C–1 Hz is hardly influenced by the optimal gradations used. Yet, the ability of blown bitumens or SBS polymers to reduce susceptibility to rutting was once again evidenced;
- secant stiffness modulus measured at 15°C–0.02 s is significantly increased (in the range of 11% to 20%). The proposed procedure to optimize aggregate packing characteristics could be used in the framework of high-modulus mix design (e.g. French ‘EME’ (enrobé à module élevé)) with slightly softer bitumen grades than usual (Penetrability at 25°C > 30), thus enhancing both reclaiming ability and fatigue resistance of such asphalt concretes;
- fatigue resistance of GB5 mixes, evaluated at 10°C–25 Hz and for a fixed considered bitumen, appears comparable to the one of the reference GB2 material. In addition, as the single or multiple gap-graded asphalt mixtures really enhanced stiffness moduli, the use of softer bitumen grades, semi-blown binders or PMBs yields great fatigue resistance. Such a particular combination of innovative single or double gap-graded curves with semi-blown and/or SBS modified bitumens, leading to very stiff and fatigue resistant polymer modified base or binder course materials in a single formulation, was recently patented by EIFFAGE Travaux Publics.

To some extent, such GB5 mixes used for base or binder courses could provide real long-life pavements that do not deteriorate structurally, needing only timely surface maintenance to maintain their overall condition.

Finally, the on-site application of GB5 mixes on many experimental roadworks with or without RAP, at either hot (160°C) or warm (125°C) or even half-warm (90°C) temperature, was successfully realized mainly in 2011: almost 120,000 tons of GB5 mixes have already been paved in France on high traffic highways and toll motorways. This rapid development and the numerous large-scale roadworks of GB5 mixes, with SBS in base or binder courses, could be put into practice, particularly because the proposed GB5 mix design relies on a rather low binder content (from 4.0 to 4.5% generally). Indeed, the optimization of aggregate packing allows obtaining such rather low binder content, and thus, makes it economically possible to use SBS in base courses, leading to an alternative concept of perpetual pavement.

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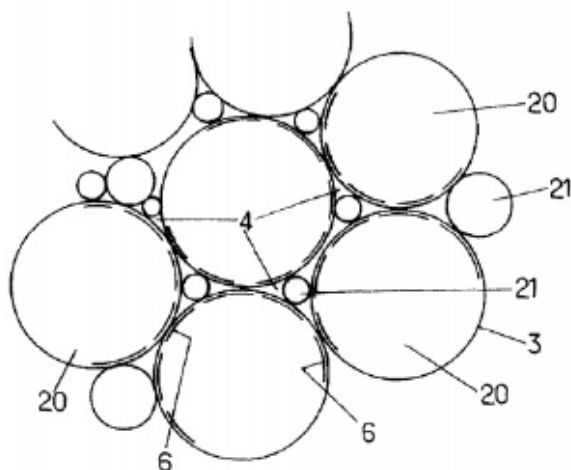


FIG.2.

(57) Abstract: The invention relates to coated material (1) for the base layer (5) of road pavement, made up of aggregate (2) coated with a hydrocarbon binder (3), wherein the aggregate (2) is more than 95 wt % of the coated material (1); wherein the aggregate (2) includes a granular structure, which includes a plurality of granular fractions  $d/D$ ; one intermediate fraction of which is less than 15% of the granules; wherein the hydrocarbon binder (3) is less than 5 wt % of the coated material; wherein the coated material (1) includes, after compacting, a void fraction of less than 8%; wherein the hydrocarbon binder (3) is a hydrocarbon binder modified by adding polymers or oil, or modified by foaming or by emulsion, by means of which the modulus of rigidity of the coated material (1), once compacted, is higher than 9000 MPa.

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CLAIMS

1. Coated material (1) for a base layer (5) or binder layer of a road or highway pavement, or for industrial, port, or airport platforms, or for a supporting layer for railroad tracks,

- wherein said coated material (1) is composed of aggregate (2) mixed with at least one hydrocarbon binder (3),

10 - wherein the aggregate (2) represents more than 95% by weight of the coated material (1), and the hydrocarbon binder (3) represents at most 5%,

- wherein the aggregate (2) comprises a granular structure comprising several particle size fractions  $d/D$ , each particle size fraction being defined by a lower limit (d) and an upper limit (D),

15 - wherein the aggregate (2) comprises a first particle size fraction  $d_1/D_1$  (11) having as median a first median  $dm_1$ , and a second particle size fraction  $d_2/D_2$  (12) having as median a second median  $dm_2$ ,

20 - wherein the aggregate comprises a third particle size fraction  $d_3/D_3$  (13) between the first and second particle size fractions, having as lower limit  $d_3$  the upper limit  $D_1$  of the first particle size fraction, and having as upper limit  $D_3$  the lower limit  $d_2$  of the second particle size fraction,

- wherein the third particle size fraction (13) has a ratio (P3) of its weight relative to the weight of the aggregate (2),

30 - wherein the width of the third particle size fraction  $D_3-d_3$ , defining a relative width  $(D_3-d_3)/D_2$  in relation to the upper limit (D2) of the second particle size fraction, said relative width being greater than 20%

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of D2,

- wherein the ratio of the weight ratio (P3) of the third particle size fraction (13) and its relative width is less than 0.4, which is:

$$5 \quad \frac{P3}{(D3-d3)/D2} \leq 0.4$$

by means of which the number of contacts between the particles of the second particle size fraction d2/D2 (12) is maximized,

- wherein the coated material (1) comprises, after  
10 compacting, a void content of less than 10%, possibly less than 8%, and preferably less than 6%,

- wherein the hydrocarbon binder (3) is a hydrocarbon binder modified by inclusion of polymers and/or oil, and/or treated by blowing and/or treated by foaming or by  
15 emulsion,

by means of which the modulus of rigidity of the coated material (1), once compacted, is greater than 9000 MPa at a temperature of 15°C and at a frequency of 10Hz, and the fatigue resistance of the coated material, once  
20 compacted, is greater than 90 microstrain at a temperature of 10°C and at a frequency of 25Hz.

2. Coated material (1) according to claim 1, wherein the ratio between the first median dm1 and the second median dm2 is less than 0.33, and preferably less than  
25 0.25.

3. Coated material (1) according to claim 1, wherein the width of the third particle size fraction D3-d3 is greater than 30% of D2-d1, and preferably greater than 40%.

30 4. Coated material (1) according to claim 1, wherein the ratio between the weight ratio (P3) of the third particle size fraction (13) and its relative width is

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less than 0.25, which is:

$$\frac{P3}{(D3-d3)/D2} \leq 0.25$$

5. Coated material (1) according to claim 4, wherein the ratio between the weight ratio (P3) of the third particle size fraction (13) and its relative width is greater than 0.10, which is:

$$\frac{P3}{(D3-d3)/D2} \geq 0.10$$

6. Coated material (1) according to claim 1, wherein the hydrocarbon binder (3) has a needle penetration depth, measured at 25°C as defined in standard EN 1426, that is greater than 30 tenths of a mm.

7. Coated material according to claim 1, wherein the fatigue resistance of the coated material, once compacted, measured at a temperature of 10°C and at a frequency of 25Hz according to standard NF EN12697-24, is greater than 110 microstrain and is preferably greater than 130 microstrain.

8. Coated material according to claim 1, wherein the modulus of rigidity of the coated material, once compacted, measured at a temperature of 15°C and at a frequency of 10Hz according to standard NF EN12697-26, is greater than 11000 MPa and is preferably greater than 14000 MPa.

9. Coated material according to claim 1, wherein the hydrocarbon binder (3) is without fibers.

10. Coated material (1) according to claim 1, additionally comprising a fourth particle size fraction  $d4/D4$  (14) and a fifth particle size fraction  $d5/D5$  (15) between the second and fourth particle size fractions, having for lower limit  $d5$  the upper limit  $D2$  of the second particle size fraction, and having for upper limit  $D5$  the lower limit  $d4$  of the fourth particle size

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fraction, wherein the width of the fifth particle size fraction is greater than 20% of the upper limit D4, wherein the fifth particle size fraction (15) has a weight (P5) relative to the weight of the aggregate (2) such that

$$\frac{P5}{(D5-d5)/D4} \leq 0.6$$

11. Coated material according to claim 1, wherein the proportion by weight of the hydrocarbon binder (3) in the coated material (1) is at most equal to 4.5%.

10 12. Pavement comprising at least one base layer or binder layer comprising a coated material (1) according to any one of the above claims.

13. Method for producing a coated material (1) for a base layer (5) or binder layer for road or highway pavement, or for industrial, port, or airport platforms, or for a supporting layer for railroad tracks,

15 said coated material (1) being composed of aggregate (2) mixed with at least one hydrocarbon binder (3), wherein the aggregate (2) comprises a granular structure comprising several particle size fractions d/D, each particle size fraction being defined by a lower limit (d) and an upper limit (D),

20 said method comprising the following steps, in any order:

25 a- providing particles of a first particle size fraction d1/D1 (11),

b- providing particles of a second particle size fraction d2/D2 (12),

30 said first and second particle size fractions (11,12) being separated by a third particle size fraction d3/D3 (13) having as lower limit d3 the upper limit D1 of the first particle size fraction, and having as upper limit

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D3 the lower limit d2 of the second particle size fraction, wherein the third particle size fraction (13) has a ratio (P3) of the weight relative to the weight of the aggregate (2), wherein the width of the third particle size fraction D3-d3, defining a relative width (D3-d3)/D2 in relation to the upper limit (D2) of the second particle size fraction, said relative width being greater than 20% of D2, wherein the ratio between the weight ratio (P3) of the third particle size fraction (13) and its relative width is less than 0.4, which is:

$$\frac{P3}{(D3-d3)/D2} \leq 0.4$$

c- adding new hydrocarbon binder until obtaining a total hydrocarbon binder of less than 5% by weight of the coated material (1), the hydrocarbon binder (3) being a hydrocarbon binder modified by inclusion of polymers and/or oil, and/or treated by blowing and/or treated by foaming or by emulsion,

d- mixing all this together.

14. Production method according to claim 13, wherein the first and second particle size fractions (11,12) comprise a proportion of recycled aggregate, and wherein the total hydrocarbon binder comprises a portion of new hydrocarbon binder and a portion of hydrocarbon binder issuing from recycled aggregate.

15. Production method according to claim 13, additionally comprising the following steps:

e- the coated material (1) is spread on a surface,

f- said coated material (1) is compacted,

by means of which the coated material (1) comprises a void content of less than 10%, possibly less than 8%, and preferably less than 6%, and by means of which the modulus of rigidity of the coated material (1) is greater than 9000 MPa at a temperature of 15°C and at a frequency

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of 10Hz, and the fatigue resistance of the coated material is greater than 90 microstrain at a temperature of 10°C and at a frequency of 25Hz.

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FIG.1.

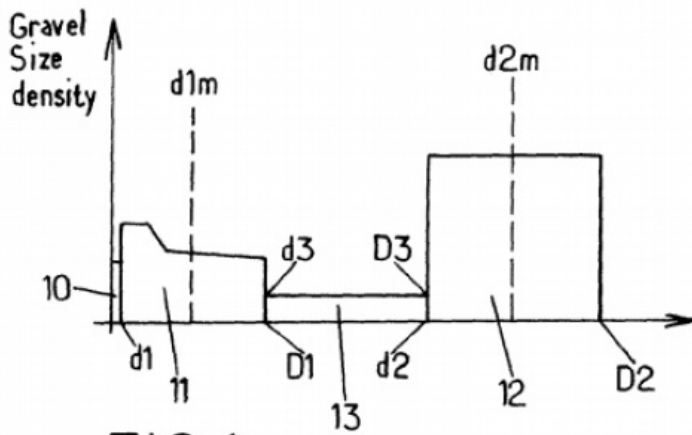
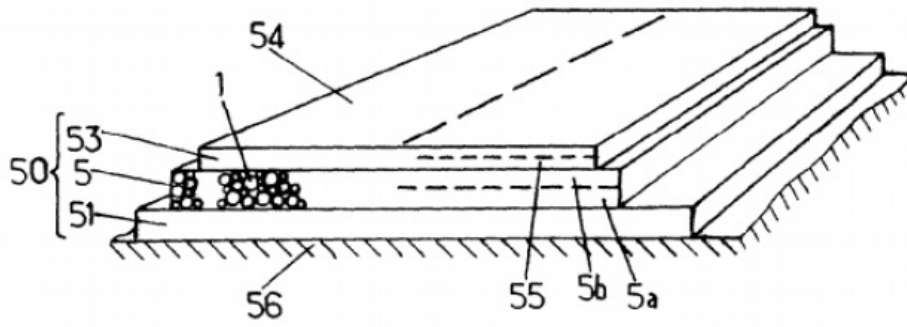


FIG.4.

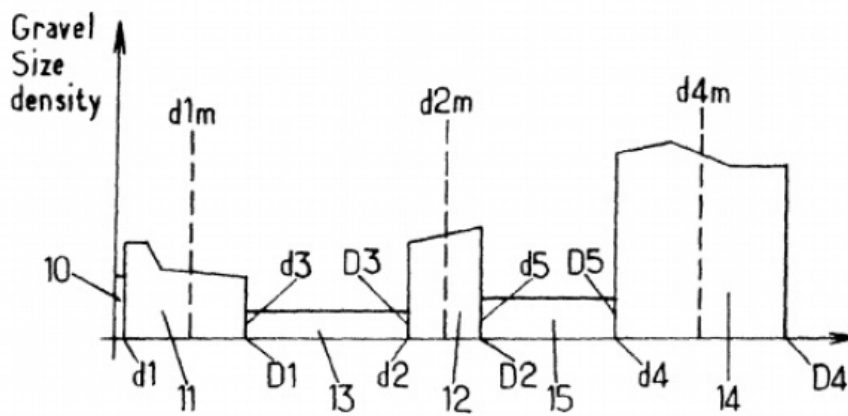


FIG.5.

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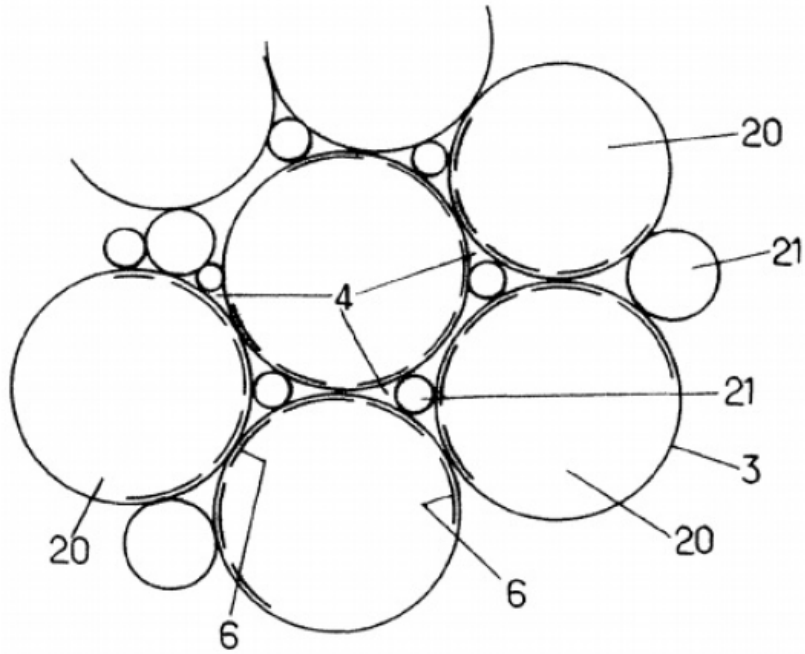


FIG. 2.

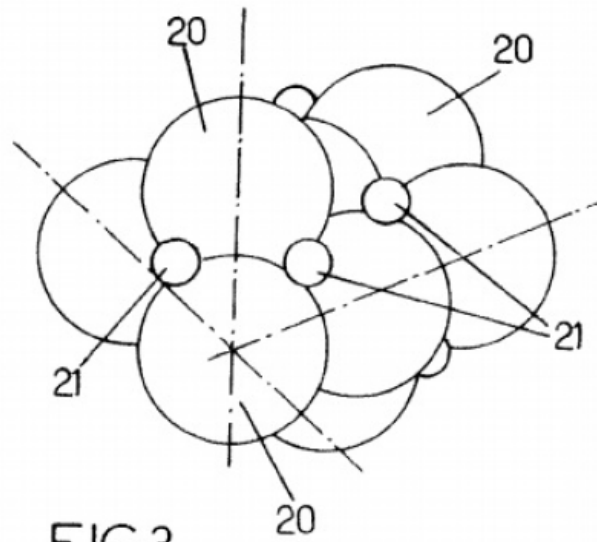


FIG. 3.





## Annexe 4

### Propriétés viscoélastiques linéaires et en fatigue des enrobés bitumineux contenant des agrégats d'enrobés recyclés (2007-2012)

Deux articles sont fournis en Annexe 4:

- F. Olard, D. Bonneau, S. Dupriet, S. Pouget, Formulation et durabilité des enrobés à fort taux de recyclés (jusqu'à 70 %), Revue Générale des Routes et des Aérodrômes, N° 902, 2012.
- S. Mangiafico, H. Di Benedetto, C. Sauzéat, F. Olard, S. Pouget, L. Planque, Influence of RAP content on complex modulus of asphalt binder blends and corresponding mixes: experimental results and modeling, Annual meeting of EATA (European Asphalt Technologists Association), Braunschweig (Germany), Special Issue of Road Materials and Pavement Design, Vol. 14, 2013 [Soumission, Journal de rang A].



## Dossier Enrobés tièdes et recyclage des enrobés

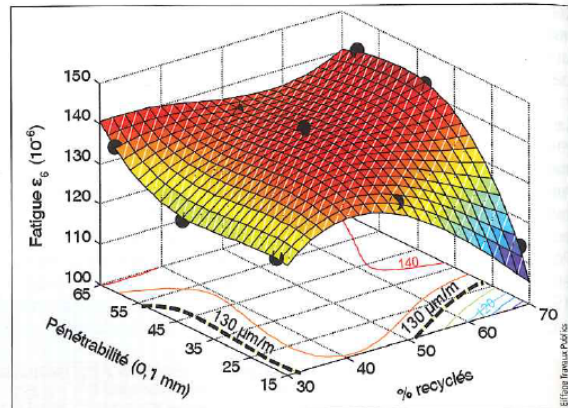
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Exemple de surface de réponse donnant l'optimum de la résistance en fatigue à 10 °C-25 Hz d'un EME 0/20 en fonction du pourcentage d'agrégats d'enrobés recyclés et de la pénétrabilité du bitume vierge d'apport.  
Example of response surface yielding optimum fatigue strength at 10°C – 25 Hz of a 0/20 high modulus asphalt (EME 0/20) as a function of percentage of recycled asphalt aggregates and penetrability of virgin bitumen used

## Formulation et durabilité des enrobés à fort taux de recyclés (jusqu'à 70 %)



Cet article présente la méthodologie d'optimisation multicritère développée aux laboratoires centraux EIFFAGE Travaux Publics de Ciry-Salsogne (02) et de Corbas (69) pour la formulation des enrobés à très fort taux de recyclés (jusqu'à 70 %). Cette stratégie expérimentale, s'appuyant sur la méthodologie des "plans d'expériences", est illustrée à travers l'exemple d'une étude d'optimisation d'un enrobé à module élevé (EME) fabriqué à chaud. La présente contribution montre tout particulièrement l'influence de la pénétrabilité du bitume d'apport et du pourcentage d'agrégats d'enrobés bitumineux sur le comportement de la formule étudiée, notamment en ce qui concerne sa résistance en fatigue et donc sa durabilité. Les résultats obtenus sont pour le moins encourageants.

### Introduction

Aujourd'hui plus que jamais, le critère d'économie de matériaux et d'économies à proprement parler est systématiquement pris en compte par les maîtres d'ouvrage, les maîtres d'œuvre et les entreprises routières. Le recyclage en centrale des enrobés bitumineux à un taux élevé est de plus en plus utilisé dans le cadre des travaux routiers et autoroutiers de réhabilitation ou de maintenance. La teneur maximale en recyclés est comprise, en général, entre 20 % et 50 % en fonction des limites matérielles des équipements de centrales considérés.

Sous l'impulsion de nombreux maîtres d'ouvrage autoroutiers et de l'entreprise Appia Grands Travaux (filiale grands travaux d'Eiffage Travaux Publics), deux procédés originaux de recyclage à fort taux – jusqu'à 50 % – sont apparus sur autoroutes françaises dès le début des années 90 :

- Le procédé RAFT® (Recyclage à fort taux d'agrégats d'enrobés), dont la 1<sup>re</sup> référence

(EME à 50 % de recyclés) date de 1994 sur le réseau APRR (A5, Langres-Chaumont). Depuis 1994, les références sur autoroutes A5 (APRR), A630 (autoroute non concédée), A7 (ASF), A10 (COFIROUTE), mais aussi A2, A26 et A29 (SANEF), s'élevèrent à plus de 500 000 m<sup>2</sup> [1].

- Le procédé RAFTED® (Recyclage à fort taux d'enrobés drainants), dont la 1<sup>re</sup> référence de béton bitumineux drainant (BBD<sup>r</sup>) à 50 % de recyclés remonte à 1993 sur la RN4 (77). Depuis 1993, les références sur RN4, autoroutes A50 (non concédée), A7 (ASF), A10 (COFIROUTE) et A43 (AREA), s'élevèrent à plus de 1 000 000 m<sup>2</sup>. Sur A43 en particulier, 900 000 m<sup>2</sup> ont été réalisés au cours des années 2003-05 (BBD<sup>r</sup> à 30 % de recyclés en 2003, 50 % en 2004 et 2005). Le procédé RAFTED® a notamment fait l'objet d'une délivrance de Charte de l'innovation ASFA en 2002 (photo 1).



Photo 1  
Charte de l'innovation ASFA de 2002 concernant le procédé RAFTED®  
ASFA 2002 Innovation Charter concerning the RAFTED® process

Les nombreuses réalisations RAFT® et RAFTED® utilisant jusqu'à 50 % de recyclés ont été possibles grâce à l'emploi d'un poste *double barrel* de technologie américaine, (ASTEC) très innovante à l'époque.

Dès 2007, une teneur en recyclés de 65 % a pu être utilisée sur chantier autoroutier, fabriqués à la centrale innovante HPR (à haut pouvoir de recyclage, Benninghoven) d'Appia Grands Travaux qui permet aussi bien le recyclage à chaud que le recyclage à tiède. Ladite centrale est décrite ci-après. Lorsqu'à la fois la ressource en agrégats d'enrobés (AE) du chantier considéré et la centrale d'enrobage utilisée permettent de planifier le réemploi d'un très fort taux d'AE recyclés (> 50 %) en couches d'assise généralement, l'étude de formulation spécifique de cet enrobé n'en demeure pas moins importante en laboratoire. Elle peut s'appuyer à bon escient sur la méthodologie des plans d'expériences factoriels et des plans de mélanges [2, 3].

Le recyclage à fort taux n'est bien évidemment pas un but en soi, il permet néanmoins, au cas par cas, d'atteindre d'autant plus rapidement les objectifs ambitieux fixés à l'occasion de la Convention d'engagement volontaire (CEV) de mars 2009.

### Description de la centrale HPR

Contrairement à l'exemple d'une centrale d'enrobage continue classique où les agrégats d'enrobés recyclés peuvent être introduits froids à hauteur de 20-30 % par l'anneau de recyclage prévu à cet effet, dans l'exemple de la centrale d'enrobage continue à haut pouvoir de recyclage (HPR), deux tambours sècheurs sont utilisés en parallèle :

- Le 1<sup>er</sup> tambour chauffe les granulats vierges sur une gamme conventionnelle de 170-250 °C.
- Le 2<sup>e</sup> tambour tiédit les agrégats d'enrobés recyclés à 130-140 °C de façon à mobiliser et à ramollir le bitume vieilli du recyclé pour atteindre *in fine* un mélange intime entre le bitume vieilli et le bitume neuf au sein du malaxeur séparé.

Les deux tambours sont alimentés respectivement par 4 et 2 trémies, via des convoyeurs (figure 1). Le poste est équipé d'un malaxeur séparé (à doubles arbres longs) en continu.

L'utilisation du tambour « tiédisseur » spécifique aux AE recyclés augmente le potentiel de recyclage jusqu'à 70 %, à la condition d'une ressource et d'une qualité en matériaux recyclés suffisantes et en fonction, bien sûr, des études de formulation réalisées au préalable en laboratoire. En effet, ce tambour « tiédisseur » chauffe à des températures modérées (130-140 °C) le recyclé et mobilise au mieux le bitume résiduel [4-6].

Cette démarche innovante s'applique parfaitement, sans s'y limiter, au cas particulier des projets de renforcement en voie lente autoroutière visant à une valorisation globale des ressources en matériaux *in situ*.

Les photos 2 et 3 illustrent la centrale HPR continue. D'autres centrales HPR, discontinues cette fois-ci, ont fait l'objet de développements similaires (photos 4).

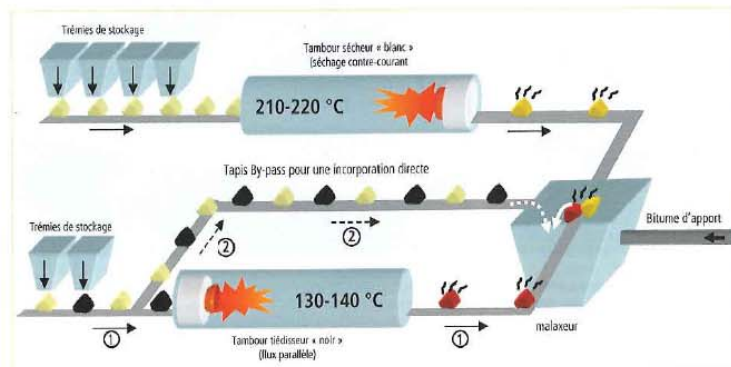


Figure 1  
Centrale continue à haut pouvoir de recyclage (HPR) d'Appia Grands Travaux ; la centrale mobile peut recycler jusqu'à 60-70 %  
High recycling capability mixing plant (HPR) of Appia Grands Travaux; the mobile plant can recycle up to 60 – 70%.



Photo 2  
Vue d'ensemble du poste HPR continu  
General view of HPR continuous mixing plant



Photo 3  
Deux tambours sècheurs en parallèle du poste HPR continu : le 1<sup>er</sup> sèche et chauffe classiquement les granulats vierges et le 2<sup>e</sup> tiédit à 130-140 °C les agrégats d'enrobés recyclés  
Two parallel drum mixers of HPR continuous mixing plant: the first dries and heats the virgin aggregates conventionally and the second warms to 130-140°C the recycled asphalt aggregates



Photos 4  
 (a) Vue d'ensemble du poste HPR discontinu, bardé, de Touraine Enrobés (Esvres-sur-Indre)  
 (b) Vue du tambour tiédisseur dédié aux agrégats d'enrobés recyclés  
 (a) General view of HPR enclosed batching plant of Touraine Enrobés (Esvres-sur-Indre)  
 (b) View of warming drum dedicated for recycled asphalt aggregate

### Objectifs de l'étude menée au laboratoire de Ciry-Salsogne

La présente étude de laboratoire vise à évaluer sur quels facteurs intervenir (taux d'agrégats, grade du bitume d'apport) pour optimiser les propriétés et la durabilité des enrobés contenant un très fort taux de recyclés. La méthode expérimentale utilisée et les principaux enseignements acquis lors de cette campagne d'essais sont décrits. L'originalité des travaux consiste en ce que les essais de fatigue à 10 °C – 25 Hz sur éprouvettes trapézoïdales ont pu être menés – malgré leur caractère long et fastidieux – afin d'évaluer la durabilité des enrobés à très fort taux de recyclés. En effet, de tels enrobés ne peuvent être simplement évalués sur des propriétés d'aptitude au compactage, de tenue à l'eau, de résistance à l'orniérage et de rigidité. L'utilisation d'AE recyclés engendre généralement une amélioration de l'ensemble de ces propriétés. C'est avant tout la notion de tenue en fatigue et donc de durabilité qui doit être évaluée.

### Plan expérimental réalisé sur un EME 0/20 à chaud

Les propriétés d'EME 0/20 de classe 2 ont été évaluées. Leurs spécificités sont rappelées dans la norme NF EN 13008-1 [7].

### Paramètres étudiés

Parmi les principaux facteurs susceptibles d'avoir une influence notable sur les propriétés des enrobés à chaud à très fort taux de recyclés, l'influence du taux de recyclage ainsi que du grade du bitume d'apport a été déterminée. Plusieurs combinaisons ont été

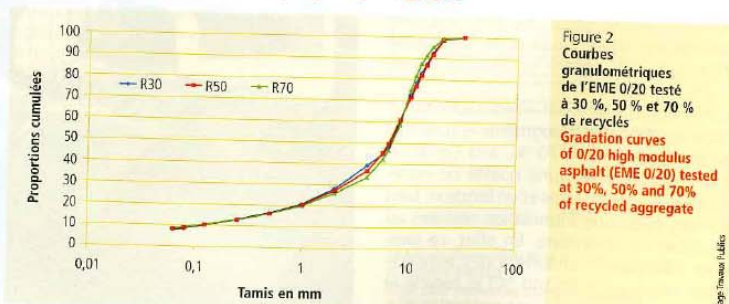
envisagées, chacun des deux facteurs étudiés prenant 3 niveaux :

- taux d'AE recyclés de 30 %, 50 % et 70 %, tiédés à 140 °C avant introduction dans le malaxeur pour se rapprocher de la production du poste HPR,
- grade du bitume d'apport (d'origine naph-ténique) : 10/20, 35/50 ou 50/70.

Teneur en recyclés	30 %	50 %*	70 %*	
10/20 Haut-Lieu	23,0 %	25,0 %	14,5 %	
6/10 Haut-Lieu	21,0 %	6,0 %		
4/6 Haut-Lieu		5,0 %	6,0 %	
0/4 Haut-Lieu	26,0 %	14,0 %	9,5 %	
0/10 Recyclés Voglans	30,0 %	50,0 %	70,0 %	
Teneur en liant d'apport	3,87 (ppc)	2,80 (ppc)	2,24 (ppc)	
Teneur totale en liant (liant vieilli + liant neuf)	5,53 (ppc)	5,60 (ppc)	5,67 (ppc)	
Courbe granulaire	6,3 mm	48,1 %	49,8 %	46,5 %
	2 mm	28,2 %	27,3 %	26,0 %
	0,25 mm	13,1 %	12,9 %	12,9 %
	0,063 mm	7,9 %	8,2 %	8,4 %
Masse vol. réelle (calculée)	2,527 g/cm <sup>3</sup>	2,485 g/cm <sup>3</sup>	2,512 g/cm <sup>3</sup>	

\* Pour travailler à squelette granulaire constant, les recyclés ont été séparés en deux coupures

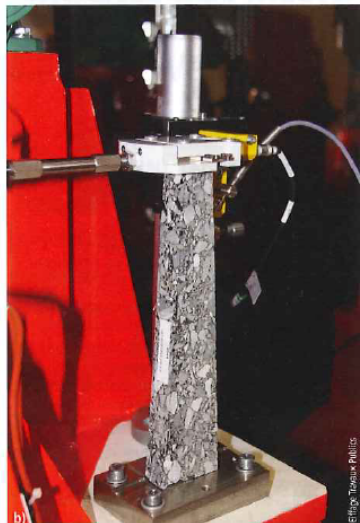
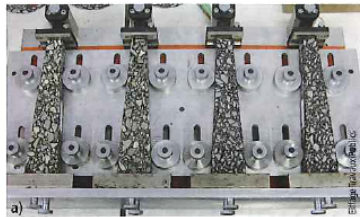
Tableau 1  
 Caractéristiques des mélanges en fonction de la quantité de recyclés  
 Characteristics of mixtures as a function of quality of recycled aggregate



### Performances obtenues

Une bonne aptitude au compactage, un module de rigidité élevé et une très bonne résistance en fatigue sont recherchés lors de la formulation des EME. Ces trois caractéristiques ont été déterminées, respectivement à partir de :

- l'essai PCG [8],
  - la mesure du module sécant sur cylindres à 15 °C et pour un temps de charge de 0,02 s [9],
  - l'essai de fatigue à 10 °C et 25 Hz sur éprouvettes trapézoïdales [10] (photos 5a et 5b).
- Pour des raisons de concision, les résultats des essais de tenue à l'eau et de résistance à l'orniérage (conformes) ne sont pas ici présentés.



Photos 5  
(a) banc de collage des casques sur les éprouvettes trapézoïdales  
(b) essai de fatigue à 10 °C et 25 Hz, en flexion deux points sur éprouvettes trapézoïdales  
(a) Bench for gluing plates on trapezoidal specimens  
(b) Fatigue test at 10 °C and 25 Hz, with two-point bending on trapezoidal specimens

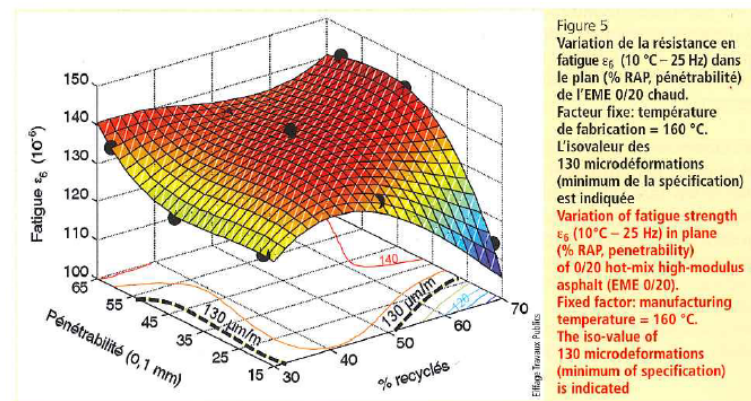
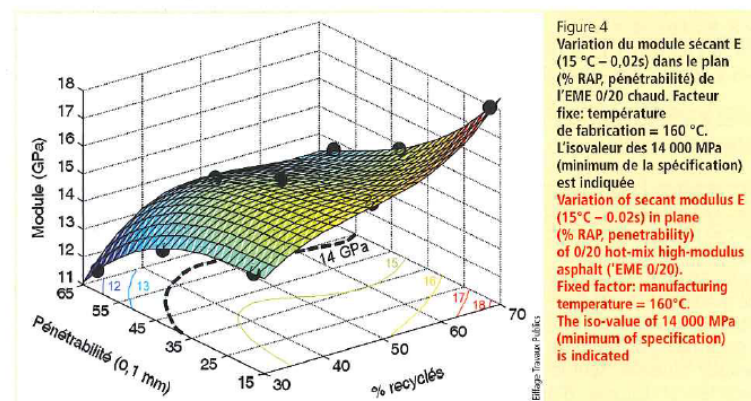
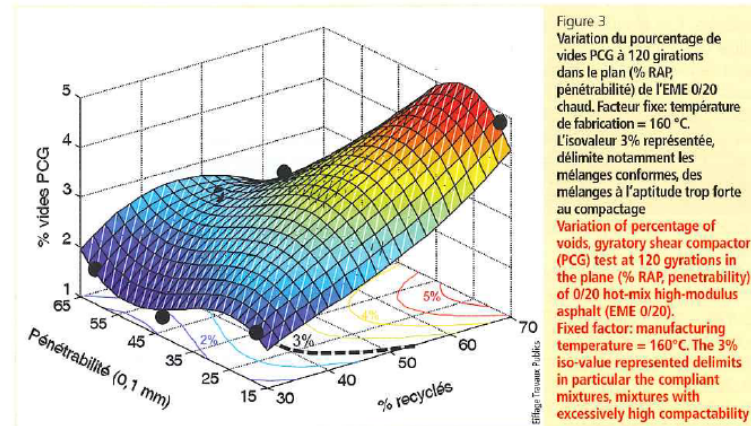
La température de fabrication, sensiblement fluctuante pour les 9 mélanges, est apparue au cours de l'analyse de données comme un facteur influent devant être pris en compte dans les calculs. Les résultats présentés ont donc été recalculés en fixant la température de fabrication à 160 °C. Les figures 3, 4 et 5 offrent une lecture 2D et 3D des résultats dans ce domaine expérimental, *i.e.* pour une pénétrabilité comprise entre 17,9 et 61,

1 1/10 mm et pour un taux de recyclés compris entre 30 et 70 %. Les figures 4 et 5 sont données pour des densités comprises entre 96 % et 97 %.

La figure 3 illustre une forte interaction entre les deux paramètres. En effet, le taux de recyclés a une très forte influence sur l'aptitude au compactage s'il est associé à un liant dur, mais il devient quasi nul lorsqu'on utilise un liant de pénétrabilité élevé. Aussi, la pénétra-

bilité du liant d'apport a peu d'influence sur le compactage à la presse à cisaillement giratoire (PCG) si le taux de recyclés est voisin de 30 %, mais elle devient très influente quand le taux de recyclage augmente.

La figure 4 montre une assez faible influence de la pénétrabilité du liant d'apport lorsque l'enrobé contient 50 à 70 % de recyclés.



Lorsque l'enrobé contient 30 à 50 % de recyclés, la diminution de la pénétrabilité du liant d'apport entraîne l'élévation du module sécant de l'enrobé. Concernant l'effet du taux de recyclés, son augmentation entraîne, comme attendu, une forte rigidification de l'enrobé (le module sécant peut atteindre 18 000 MPa).

La figure 5 illustre les résultats des essais de fatigue, où chaque point correspond aux mesures effectuées sur 18 éprouvettes trapézoïdales. Ces résultats montrent que la durabilité des enrobés est très bonne pour des taux de recyclage élevés (de 30 % à 70 %) et pour des bitumes de grades 15/25, 35/50 ou 50/70, à la seule exception du cas extrême avec 70 % de recyclés et bitume dur 15/25. L'évolution du pourcentage de vides PCG à 120 girations, du module sécant à 15 °C et de la tenue en fatigue à 10 °C, sont approchées par les polynômes de degré 3 suivants :

- % vides PCG (120 girations) :  $-2,2832E-5 Y^3 + 1,3699E-4 X^3 - 9,5441E-5 Y^2X + 7,8438E-5 YX^2 - 1,5E-3 Y^2 - 0,0179 X^2 + 2,4E-3 YX + 0 Y + 0,8965 X + 0$
- Module sécant  $E(15 °C, 0,02 s) = -2,2832E-5 Y^3 + 1,3699E-4 X^3 - 9,5441E-5 Y^2X + 7,8438E-5 YX^2 - 1,5E-3 Y^2 - 0,0179 X^2 + 2,4E-3 YX + 0 Y + 0,8965 X + 0$
- Fatigue  $\epsilon_6 : 3,9924E-4 Y^3 + 6,5738E-4 X^3 + 8,8146E-4 Y^2X - 1,03446E-3 YX^2 - 4,87873E-3 Y^2 - 0,14736911 X^2 + 5,72889E-3 YX + 0 Y + 7,819732 X + 0$

(avec  $X = \% \text{ recyclés}$  et  $Y = \text{pénétrabilité du bitume d'apport}$ )

Des résultats PCG et de module ici présentés, l'EME 0/20 calcaire fabriqué à chaud (à 160 °C) présente un optimum de formulation pour un taux de recyclés compris entre 45 et 70 % et pour un liant d'apport de grade 10/20 à 35/50.

Concernant les résultats de résistance en fatigue obtenus, seul le couple 70 % de recyclés et bitume d'apport 15/25 ne satisfait pas au seuil conventionnel des 130 microdéformations. Les huit autres combinaisons sont toutefois conformes, ce qui est très encourageant d'un point de vue à la fois technique, économique et environnemental.

Si l'ensemble de ces résultats de laboratoire ne peuvent pas être généralisés, il n'en reste pas moins que cette méthodologie d'optimisation des formulations peut être appliquée à tout type de formules d'enrobés et de recyclés.

## Conclusions et perspectives

Dans une perspective de développement durable et aussi dans un contexte haussier des prix et d'approvisionnement en ressources (bitumes et granulats) de plus en plus difficile et tendu, la centrale d'enrobage innovante HPR illustre la possibilité désormais offerte de recycler à très fort taux (jusqu'à 70 %) les enrobés bitumineux.

Hormis les équipements matériels spécifiques de ladite centrale ici détaillés, une ressource en matériaux recyclés en quantité et en qualité suffisantes est cependant nécessaire. L'ensemble de la profession routière doit, en effet, organiser en conséquence les campagnes de rabotage (ressources), les zones de traitement (préparation et stockage) et les matériels de centrales d'enrobage adéquats (adaptation de l'outil industriel).

Le contexte des travaux de réhabilitation et d'entretien des voies lentes autoroutières se prête particulièrement bien à l'utilisation de très forts taux d'agrégats d'enrobés, mais la portée de cet article ne s'y limite aucunement.

Etant donné la grande nouveauté des mélanges bitumineux contenant de si forts taux de recyclés, une méthodologie spécifique de formulation en laboratoire a été employée, faisant appel à la méthode des plans d'expériences factoriels et des plans de mélanges.

L'originalité des présents travaux consiste en ce que les essais de fatigue à 10 °C - 25 Hz sur éprouvettes trapézoïdales ont pu être menés – malgré leur caractère long et fastidieux – afin d'évaluer la durabilité de tels enrobés à très fort taux de recyclés. En effet, de tels enrobés ne peuvent être simplement évalués sur des propriétés d'aptitude au compactage, de tenue à l'eau, de résistance à l'orniérage et de rigidité. L'utilisation d'AE recyclés, à fort taux, engendre généralement une amélioration de l'ensemble de ces propriétés. En aucun cas, les études de laboratoire ne doivent se borner à des études de niveau 3 (i.e. sans la caractérisation de la résistance en fatigue). C'est avant tout la notion de tenue en fatigue (niveau 4) et donc de durabilité qui doit être évaluée.

Pour conclure, des efforts supplémentaires restent à fournir pour poursuivre et mener à bien la reproductibilité de la démarche, notamment en l'appliquant aux formulations diverses de la profession, y compris discontinues, et cela en utilisant plusieurs fractions granulométriques d'agrégats d'enrobés, si nécessaire. Sans oublier le suivi dans le temps des chantiers d'enrobés bitumineux d'assise à très fort taux de recyclés (> 55 %) déjà réalisés. ■



### Mix design and durability of very high percentage recycled asphalt (up to 70%)

This article deals with the multi-criteria optimisation methodology developed at the EIFFAGE Travaux Publics central laboratories of Ciry-Salsogne (Aisne region) and Corbas (Rhône region) for the mix design of very high percentage recycled asphalt (up to 70%). This experimental strategy, based on the methodology for the "design of experiments," is illustrated by the example of an optimisation study on a high modulus hot mix asphalt (French designation EME). The present contribution shows in particular the influence of the penetrability of the bitumen used and the percentage of asphalt aggregates on the performance of the studied mix design, in particular as regards its fatigue strength and hence its durability. The results obtained are at least encouraging.

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## **Influence of RAP content on complex modulus of asphalt binder blends and corresponding mixes: experimental results and modeling**

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*ABSTRACT. The objective of the presented study is to determine LVE properties of corresponding binders and mixes and to check how they change with RAP content. The investigation is part of a wider ongoing research project, in the framework of a PhD thesis, in collaboration between Université de Lyon/ENTPE, EIFFAGE Travaux Publics and Beyond Petroleum (BP). Dynamic Shear Rheometer (DSR) and tension/compression (using a Métravib device) complex modulus tests were performed on nine different bitumens, produced as blends of two different base bitumens and RAP-extracted bitumen in various proportions. LVE properties of six asphalt mixes, produced with the same materials and proportions of certain bitumen blends among the nine tested ones, were measured in tension/compression mode. 2S2PID model (2 Springs, 2 Parabolic Elements, 1 Dashpot) was used to fit experimental data both for binders and mixes. SHStS (Shift-Homothety-Shift in time-Shift) transformation (developed at ENTPE) was applied to verify the correspondence of LVE behaviors of related binders and mixes.*

*KEYWORDS: complex modulus, RAP, blending, 2S2PID, SHStS, asphalt.*

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## 1. Introduction

Sustainability reasons make Reclaimed Asphalt Pavement (RAP) material a necessary solution for asphalt industry. Research efforts must offer theoretical and technological advancements to support and foster its use. Particularly relevant issues are the needs to increase recycling rates, to characterize and model the influence of RAP material on properties of mixes produced with recycled material, to determine the usable residual asphalt binder within RAP and to predict mix linear viscoelastic (LVE) behavior from binder properties of fresh and aged RAP binder (and vice versa).

The objective of the presented study is to determine LVE properties of corresponding binders and mixes and to check how they change with RAP content. In order to accomplish this, 2S2P1D (2 Springs, 2 Parabolic elements, 1 Dashpot) model and SHStS (Shift-Homothety-Shift in time-Shift) transformation, both developed at the École Nationale Travaux Publics de l'État (ENTPE), were used. The investigation is part of a wider ongoing research project, in the framework of a PhD thesis, in collaboration between Université de Lyon/ENTPE, EIFFAGE Travaux Publics and Beyond Petroleum (BP).

An extensive experimental campaign developed for this study consists of nine different bitumens, produced as blends of two different base bitumens and RAP-extracted bitumen in various proportions, and six asphalt mixes, produced with the same proportions of certain bitumen blends among the nine tested ones. Tests on binders were conducted using a Dynamic Shear Rheometer (DSR) and a tension/compression device. Tension/compression tests were also performed on mixes. LVE properties of all materials were fitted with the 2S2P1D (2 Springs, 2 Parabolic Elements, 1 Dashpot) model. As already pointed out, SHStS transformation was applied to verify the correspondence of LVE behaviors of related binders and mixes.

Specific attention is given to this point in order to understand whether, and to what extent, the assumption of blending between fresh and RAP binder in the mix can be confirmed.

## 2. Literature background

### 2.1. Influence of RAP on mechanical properties of asphalt mixes

Various studies have been conducted in order to evaluate the influence of RAP content on mechanical properties of mixes. In general, the addition of RAP material is observed to cause a significant increase of complex modulus of mixes (Li *et al.*, 2008; Huang *et al.*, 2005, Tapsoba *et al.*, 2012). The increase becomes more important with higher percentages of RAP. Unfortunately, research on mixes produced with high percentages of RAP is still limited, although developing

(Mangiafico *et al.*, 2012a and 2013; Olard *et al.*, 2008, 2009a, 2009b and 2012; Doyle *et al.*, 2011).

A non-negligible aspect seems to be the degree of blending of binder in RAP material with fresh added binder in the final mix. As a first approximation, research on this subject showed that for low contents of RAP the occurrence of actual blending could be neglected, while for high contents of RAP at least a certain amount of RAP binder blends with the fresh binder (McDaniel *et al.*, 2000; Stephens *et al.*, 2001; Huang *et al.*, 2005; Bonaquist, 2007; Al-Qadi *et al.*, 2009).

## 2.2. Modeling of mechanical behavior of bitumen blends

If the hypothesis of RAP binder and fresh binder actually blending within the mix is assumed, mechanical properties of bitumen blends must be characterized and modeled. Characterization and prediction of viscoelastic properties of bitumen blends is traditionally performed by using blending charts (Al-Qadi *et al.*, 2007; Swiertz *et al.*, 2011) or totally empirical laws, such as the so-called "log-log rule" (EN 13108-1:2006 + AC:2008). Recent studies tried to develop a more rational approach to the subject, by proposing a method to estimate the viscoelastic behavior of blends over the whole temperature and frequency range from properties of fresh and RAP binder constituents (Mangiafico *et al.*, 2012b; Mangiafico *et al.*, 2012c).

## 2.3. Prediction of mechanical behavior of asphalt mixes from binder properties

Various authors have studied the dependence of mechanical properties of asphalt mixes, especially complex modulus and fatigue resistance, on composition and mix design parameters. Predictive equations have been developed to estimate complex modulus of mixes from binder properties: very widely used are Francken's (Francken *et al.*, 1995 and 1996), Hirsch's (Christensen *et al.*, 2003) and Witczak's (Witczak *et al.*, 1996; Bari *et al.*, 2006) models. They are based on the dependency of complex modulus on, among other parameters, binder modulus or viscosity and granular size distribution.

Another approach consists in using rheological analogical model. A complete framework including development of linear viscoelastic (LVE) 2S2P1D model and prediction of mix LVE behavior from binder LVE properties (and vice et versa) (Olard *et al.*, 2003b and 2005, Di Benedetto *et al.*, 2004, 2007 and 2009, Delaporte *et al.* 2009, Pouget *et al.*, 2010) was developed at Université de Lyon/ENTPE. Both analytical tools are used in this study and are described in sections 3.3 and 3.4.

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### 3. Materials and methods

#### 3.1. Materials

Six asphalt mixes were produced, by mixing RAP material with new aggregates and two different fresh binders (35-50 and 70-100 penetration grades) in various proportions (0%, 20% and 60%).

All mixes have the following common characteristics:

- fixed continuous 0/14 gradation curve, HiMA type (Figure 1);
- silica-limestone aggregate, obtained from a single lot;
- RAP material obtained from a single lot;
- 5.35% total binder content by weight of the final mix;
- void content targeted at  $5.0\% \pm 1.5\%$ .

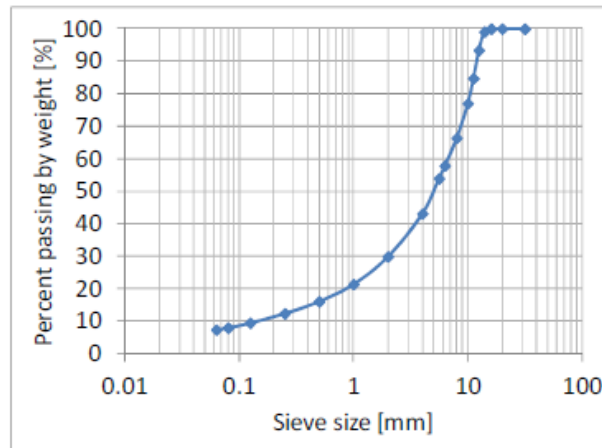


Figure 1. Continuous 0/14 gradation curve used for mixes

It has to be underlined that RAP content percentages were calculated by weight of the total mix without fresh binder (weight of aggregates and RAP material): therefore 0%, 20%, 40% and 60% correspond, respectively, to 0%, 18.7%, 37.8% and 57.0% by weight of the total mix. For simplicity, hereafter RAP contents are referred to as 0%, 20%, 40% and 60% (Table1).

**Table 1.** *Tested mixes*

Mix	RAP content	Fresh binder
35-50	0.0%	35-50
35-50 + 20% RAP	18.7%	
35-50 + 60% RAP	37.8%	
70-100	0.0%	70-100
70-100 + 20% RAP	18.7%	
70-100 + 60% RAP	37.8%	

For all mixes, a fully blended binder was reproduced by mixing fresh and RAP-extracted binder (EN 12697-3:2006) with the same proportions. This blended binder was used to fix production temperatures of mixes. They are equal to equiviscous temperatures (with a 10°C tolerance), following European norm EN 12697-35:2004+A1:2007. In any case, production temperature was limited to 185°C.

Both 35-50 and 70-100 binders are straight run binders and come from BP Lavéra refinery (France). Binder blends containing 40% RAP binder were also produced using both pure bitumen bases. A total of nine binder (including RAP binder) was therefore considered in the experimental campaign. A summary table for all binders is reported in Table 2, including results of classical semi-empirical tests (penetration, EN 1426:2007; Ring-Ball temperature, EN 1427:2007; Fraas, EN 12593:2007) and SARA (Saturates, Aromates, Resins and Asphaltenes) fractioning.

**Table 2.** *Binder characteristics [BP laboratory, Gelsenkirchen, Germany]*

Bitumen	Penetration [10 <sup>-4</sup> m]	T <sub>R&amp;B</sub> [°C]	T <sub>Fraass</sub> [°C]	SARA			
				Saturates [%]	Aromates [%]	Resins [%]	Asphaltenes [%]
35-50	34	54.2	-7	5.2	59.4	24.8	10.7
35-50 + 20% RAP	24	57.8	-7	5.9	57.9	24.5	11.8
35-50 + 40% RAP	19	61.0	-6	4.7	53.4	27.5	14.4
35-50 + 60% RAP	16	64.8	-1	4.7	50.5	29.4	15.3
70-100	73	47.4	-14	6.5	59.7	24.7	9.1
70-100 + 20% RAP	42	51.2	-9	6.8	58.2	24.5	10.6
70-100 + 40% RAP	33	55.8	-11	5.5	53.8	29.0	11.8
70-100 + 60% RAP	22	60.4	-5	5.1	51.7	29.8	13.4
RAP binder	9	72.0	1	3.6	44.2	34.1	18.1

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### 3.2. Experimental procedures

Complex modulus tests were performed on all bitumens using a Dynamic Shear Rheometer apparatus (DSR), at temperatures ranging from  $-16.5^{\circ}\text{C}$  to  $+70^{\circ}\text{C}$  and over a whole range of frequencies ranging from 0.01 Hz to 30 Hz (Figure 2). A 8 mm diameter plate-plate configuration (2 mm gap) was used. Complex modulus  $E^*$  was calculated from measured shear complex modulus  $G^*$  by applying an arbitrary Poisson's ratio equal to 0.5 ( $E^* = 3G^*$ ).



**Figure 2.** (left) DSR apparatus used for bitumen tests; (right) Close-up on a test sample [EIFFAGE Travaux Publics laboratory, Corbas, France]

Five binders (35-50 binder, RAP-extracted binder and the three deriving blends) were also tested using a Métravib apparatus, in tension/compression mode, at temperatures ranging from  $-30^{\circ}\text{C}$  to  $+10^{\circ}\text{C}$  and at frequencies ranging from 0.01 Hz to 30 Hz. Sample geometry consists of a 20 mm long cylinder with a 10 mm diameter. Figure 3 shows a picture of a Métravib.



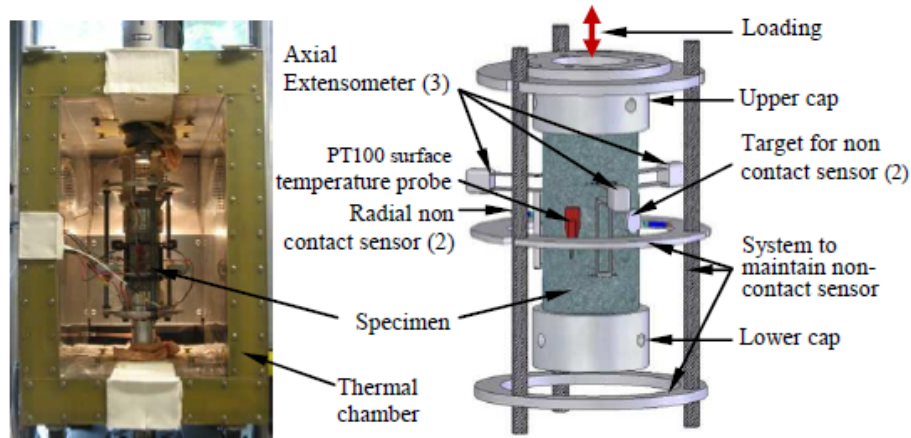
**Figure 3.** *Métravib apparatus used for bitumen tests [EIFFAGE Travaux Publics laboratory, Corbas, France]*

Complex modulus tests on mixes were performed in tension/compression configuration on cylindrical samples (150 mm high with a 75 mm diameter). Samples were cored from slabs compacted using a LCPC-type roller compactor, according to EN 12697-33:2003+A1:2007 standard. Void content of each sample was measured. Cyclic sinusoidal loading was performed using a hydraulic press, as shown in Figure 4, in strain-controlled mode. Axial stress was measured with a load cell, while axial strain was obtained from three extensometers placed at a 120° distance and radial strain was deduced from non-contact transducers. Samples were loaded at nine different frequencies (from 0.001 Hz to 10 Hz) and seven temperatures (from -25°C to +40°C).

Table 3 shows a summary of all tests performed on all materials.



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**Figure 4.** (left) Tension-compression test apparatus used for tests on asphalt mixes; (right) Detailed scheme of apparatus and sample (Tapsoba, 2012) [ENTPE laboratory, Vaulx-en-Velin, France]

**Table 3.** Summary of all tests performed on all materials

Binders	DSR	Métravib	Mixes	T/C
35-50	X	X	35-50	X
35-50 + 20% RAP	X	X	35-50 + 20% RAP	X
35-50 + 40% RAP	X	X		
35-50 + 60% RAP	X	X	35-50 + 60% RAP	X
70-100	X		70-100	X
70-100 + 20% RAP	X		70-100 + 20% RAP	X
70-100 + 40% RAP	X			
70-100 + 60% RAP	X		70-100 + 60% RAP	X
RAP binder	X			

Test	Temperatures	Frequencies
DSR	-16.5°C → +70°C	0.01 Hz → 30 Hz
Métravib T/C	-30°C → +10°C	0.01 Hz → 30 Hz
T/C	-25°C → +40°C	0.001 Hz → 10 Hz

### 3.3. 2S2P1D LVE model

Experimental results were analyzed using 2S2P1D model (Olard *et al.*, 2003b; Olard *et al.*, 2005; Di Benedetto *et al.*, 2004; Pouget *et al.*, 2010; Tiouajni, 2011),

developed at ENTPE. It consists of two elastic (spring) elements, two parabolic creep elements and one purely viscous (dashpot) element (an analogical representation is shown in Figure 5, top). The value of complex modulus given by the model is given in Equation 1:

$$E^*(i\omega\tau) = E_{00} + \frac{E_0 - E_{00}}{1 + \delta(i\omega\tau)^{-k} + (i\omega\tau)^{-h} + (i\omega\beta\tau)^{-1}} \quad [1]$$

where:

$\omega$  is the pulsation, so that  $\omega/2\pi$  is equal to the frequency  $f$ ;

$\tau$  is the characteristic time, function only of temperature and accounting for the Time Temperature Superposition Principle (TTSP);

$E_0$  is the value of the complex modulus when  $\omega\tau \rightarrow \infty$ ;

$E_{00}$  is the value of the complex modulus when  $\omega\tau \rightarrow 0$ , equal to 0 in the case of pure bitumens;

$\delta$ ,  $k$  and  $h$  are dimensionless constants;

$\beta$  is a dimensionless parameter, related to Newtonian viscosity  $\eta$  by

$$\eta = (E_0 - E_{\infty})\beta\tau \quad [2]$$

Figure 5 (bottom) shows the meaning of the seven constants ( $E_0$ ,  $E_{00}$ ,  $\delta$ ,  $k$ ,  $h$  and  $\beta$ ) needed to describe the linear viscoelastic behavior of the material on a general Cole-Cole curve of a bitumen. Characteristic time  $\tau$  depends on temperature as follows:

$$a_T = \frac{\tau}{\tau_0} \quad [3]$$

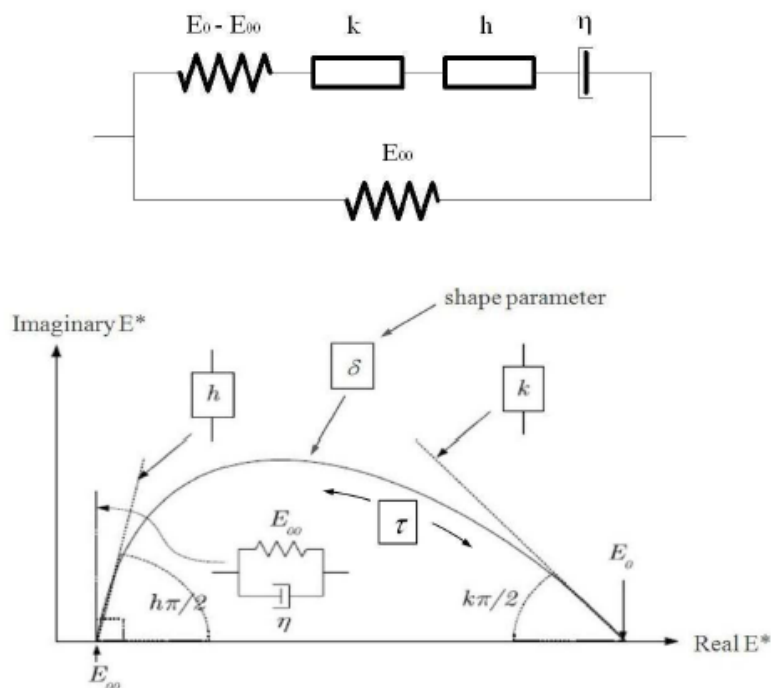
where  $\tau_0$  is the characteristic time at reference temperature  $T_{ref}$  and  $a_T$  is the shift factor, varying with temperature.  $a_T$  values may be approximated by the Williams-Landel-Ferry (WLF) equation (Ferry, 1980):

$$\log(a_T) = \frac{-C_1(T - T_{ref})}{C_2 + (T - T_{ref})} \quad [4]$$

where  $C_1$  and  $C_2$  are empirical constants depending on the material.

Experimental data were modeled using 2S2P1D model: complex modulus master curves ( $T_{ref} = 15^\circ\text{C}$ ) and Cole-Cole curves were built for all materials.

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**Figure 5.** (top) Analogical representation of 2S2P1D model (constitutive elements and associated parameters); (bottom) Meaning of 2S2P1D parameters associated with constitutive elements of the model on a general Cole-Cole curve of a bitumen

### 3.4. SHStS transformation

Studies carried out at the ENTPE showed that the behavior of bituminous materials (such as asphalt mixes or mastics) is directly dependent on the behavior of the bituminous binders used to produce them, according to their aggregate structure (Olard *et al.*, 2003b; Di Benedetto *et al.*, 2004). It is useful to introduce the normalized complex modulus  $E_{\text{norm}}^*$ , as expressed by

$$E_{\text{norm}}^* = \frac{E^* - E_{00}}{E_0 - E_{00}} \quad [5]$$

where  $E_{00}$  and  $E_0$  are, respectively, the minimum and the maximum asymptotic values of the norm of complex modulus.

When plotted in a Cole-Cole diagram, normalized moduli of materials produced with the same binder generate curves that superimpose with each other. This confirms that time and temperature dependency of these materials (which underlies the concept of TTSP) originates from the binder behavior, regardless of the

aggregate skeleton. It is possible to predict the LVE behavior of an asphalt mix from the LVE properties of the corresponding binder and vice versa. On a common Cole-Cole plan, the relationship corresponds to the sequence of a shift along the horizontal axis, a homothetic transformation, a shift of characteristic time and a second shift along the horizontal axis. For this reason, it is called SHStS (Shift-Homothety-Shift in time-Shift) transformation (Pouget *et al.*, 2010 and 2011). This relationship is independent of any rheological model and it allows calculating the mix complex modulus at temperature T if binder modulus is known (and vice-versa) at the same temperature. The equation describing the relationship is:

$$E_{\text{mix}}^*(\omega, T) = E_{00, \text{mix}} + \left[ E_{\text{binder}}^*(10^\alpha \omega, T) - E_{00, \text{binder}} \right] \frac{E_{0, \text{mix}} - E_{00, \text{mix}}}{E_{0, \text{binder}} - E_{00, \text{binder}}} \quad [6]$$

where parameter  $\alpha$  depends on the considered mix design.

Since  $E_{00, \text{binder}}$  is nil for pure binders, a simplified formula can be considered for mixes produced with them:

$$E_{\text{mix}}^*(\omega, T) = E_{00, \text{mix}} + \left[ E_{\text{binder}}^*(10^\alpha \omega, T) \right] \frac{E_{0, \text{mix}} - E_{00, \text{mix}}}{E_{0, \text{binder}}} \quad [7]$$

If TTSP is verified for binders, the following property can be added for both binders and mixes:

$$E^*(\omega, T) = E^*[\omega a_T(T), T_{\text{ref}}] \quad [8]$$

#### 4. Results

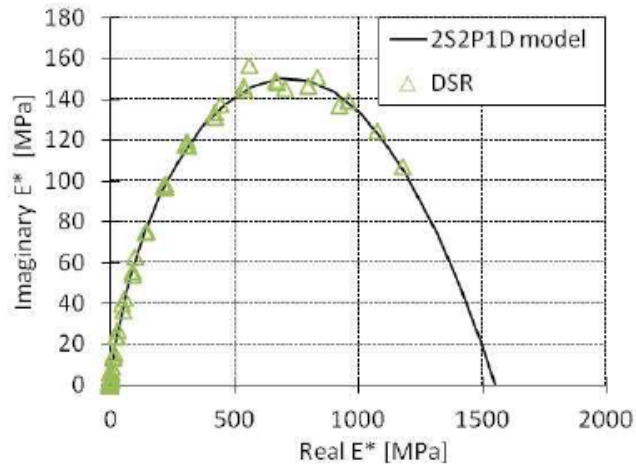
DSR complex modulus test results for all binders were fitted with 2S2P1D model. Figure 6 shows an example of model calibration for binder 70-100 + 20% RAP on a Cole-Cole plan. 2S2P1D parameters obtained for all nine binders are reported in Table 4.

Resulting  $E_0$  values appear lower than expected. In fact, standard literature values are approximately comprised between 2.1 to 3 GPa for all bitumens (Olard *et al.*, 2003a), while obtained experimental data show values lower than 1.9 GPa. It is suspected that these values are not correct because of experimental limitations of the DSR device at low temperatures.

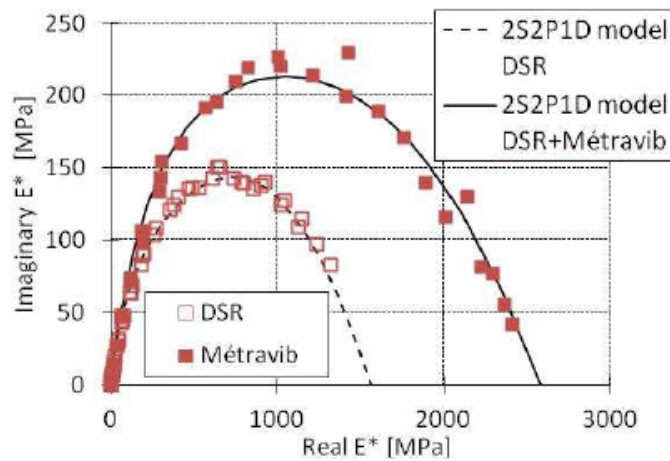
For this reason, Métravib tests were performed on bitumen blends of 35-50 and RAP-extracted binder, at temperatures ranging from -30°C to +10°C and at frequencies from 0.01 Hz to 30 Hz. Métravib data were used to replace DSR data obtained at temperatures lower than +10°C: thus, new DSR+Métravib sets of data

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were obtained and 2S2P1D model was fitted. An example of model calibration for binder 35-50 + 20% RAP in shown in Figure 7. 2S2P1D parameters obtained for the five blends from DSR+Métravib data are reported in Table 5.



**Figure 6.** Example of 2S2P1D model fitting for binder 70-100 + 20% RAP tested with DSR apparatus (from  $-16.5^{\circ}\text{C}$  to  $+60^{\circ}\text{C}$  and from 0.01 Hz to 30 Hz)



**Figure 7.** Example of 2S2P1D model fitting for binder 35-50 + 20% RAP tested with DSR (from  $-16.5^{\circ}\text{C}$  to  $+70^{\circ}\text{C}$  and from 0.01 Hz to 30 Hz) and both DSR (from  $+10^{\circ}\text{C}$  to  $+70^{\circ}\text{C}$  and from 0.01 Hz to 30 Hz) and Métravib apparatus (from  $-30^{\circ}\text{C}$  to  $+10^{\circ}\text{C}$  and from 0.01 Hz to 30 Hz)

**Table 4.** 2S2P1D parameters for binders tested with DSR apparatus (from -16.5°C to +70°C and from 0.01 Hz to 30 Hz)

bitumen blend	$E_{00}$ [MPa]	$E_0$ [MPa]	k	h	$\delta$	$\tau_0$ [s]	$\beta$
15/25	0	1500	0.220	0.580	4.50	1.00E-03	750
15/25 + 20% RAP	0	1780	0.220	0.580	4.70	1.70E-03	700
15/25 + 40% RAP	0	1640	0.220	0.585	4.80	2.20E-03	550
15/25 + 60% RAP	0	1640	0.220	0.590	4.90	4.50E-03	450
35/50	0	1550	0.225	0.590	3.80	1.60E-04	230
35/50 + 20% RAP	0	1560	0.223	0.595	4.00	4.50E-04	250
35/50 + 40% RAP	0	1710	0.223	0.600	4.45	1.25E-03	250
35/50 + 60% RAP	0	1630	0.222	0.600	4.65	2.40E-03	250
70/100	0	1630	0.235	0.575	3.40	1.50E-05	250
70/100 + 20% RAP	0	1550	0.232	0.580	3.80	7.30E-05	250
70/100 + 40% RAP	0	1620	0.230	0.590	4.10	2.20E-04	250
70/100 + 60% RAP	0	1810	0.227	0.600	4.50	5.50E-04	250
RAP bitumen	0	1830	0.221	0.610	5.20	1.10E-02	220

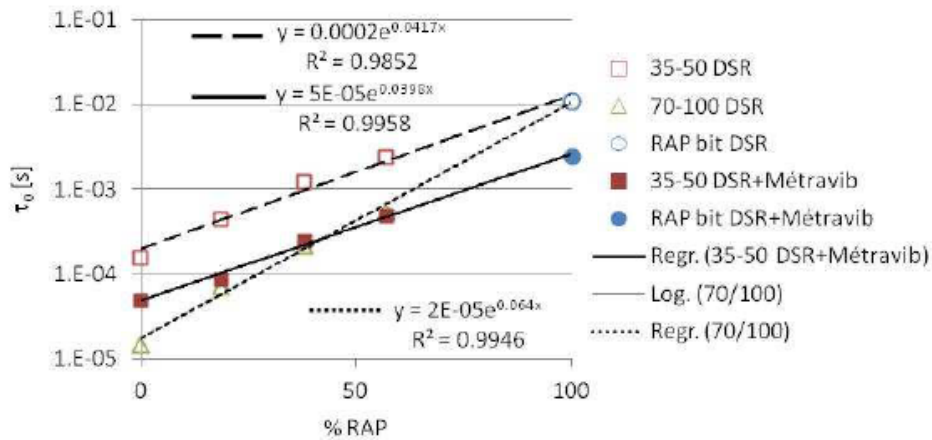
**Table 5.** 2S2P1D parameters for binders tested with both DSR (from +10 °C to +70°C) and Métravib apparatus (from -30 °C to +10°C)

bitumen blend	$E_{00}$ [MPa]	$E_0$ [MPa]	k	h	$\delta$	$\tau_0$ [s]	$\beta$
35/50	0	2450	0.191	0.539	2.85	5.00E-05	700
35/50 + 20% RAP	0	2580	0.190	0.540	2.90	9.00E-05	800
35/50 + 40% RAP	0	2650	0.189	0.530	3.40	2.50E-04	1000
35/50 + 60% RAP	0	2740	0.188	0.528	3.25	5.00E-04	1500
RAP bitumen	0	2750	0.186	0.520	3.45	2.50E-03	1700

A non negligible difference is observed between DSR+Métravib and DSR-only data. 2S2P1D model fits satisfactorily on both set of data, although different set of parameters ensue.  $E_0$  parameters estimated by using DSR+Métravib data appear to be more coherent with literature.

Previous research carried out by the authors showed a clear evolution pattern of 2S2P1D parameters of bitumen blends with RAP binder content (Mangiafico *et al.*, 2012b and 2012c). Such concept is applied for this study as well. Figure 8 shows the evolution of parameter  $\tau_0$  with RAP binder content for all tested binder blends. Both DSR-only and DSR+Métravib results are shown for binder blends produced with 35-50 bitumen, while only DSR results are available for blends produced with 70-100 bitumen.

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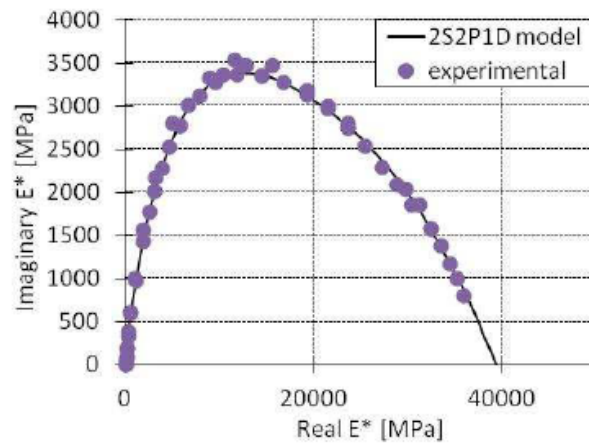


**Figure 8.**  $\tau_0$  parameter evolution with RAP material content for binders tested with DSR apparatus (from  $-16.5^\circ\text{C}$  to  $+70^\circ\text{C}$  and from  $0.01\text{ Hz}$  to  $30\text{ Hz}$ ) and both DSR (from  $+10^\circ\text{C}$  to  $+70^\circ\text{C}$  and from  $0.01\text{ Hz}$  to  $30\text{ Hz}$ ) and Métravib apparatus (from  $-30^\circ\text{C}$  to  $+10^\circ\text{C}$  and from  $0.01\text{ Hz}$  to  $30\text{ Hz}$ )

As already shown in previous studies, 2S2P1D parameters of blends appear to follow a linear trend with RAP binder content. In particular,  $\tau_0$  values resulting from DSR tests tend toward the unique value of RAP binder. The same tendency is observed for results of DSR+Métravib tests on 35-50 blends, although the corresponding fitting line does not coincide with the one obtained for DSR-only data. DSR+Métravib data show provide  $\tau_0$  values lower than DSR-only data while exhibiting an approximately parallel trend.

2S2P1D model was fitted on data obtained from tests on asphalt mixes as well. Figure 9 shows an example for mix produced with 70-100 binder and 20% of RAP material. 2S2P1D parameters for all mixes are reported in Table 6. The model accurately fits experimental data.

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**Figure 9.** Example of 2S2P1D model fitting for mix 70-100 + 20% RAP tested in T/C mode (from  $-24.9^{\circ}\text{C}$  to  $+38.8^{\circ}\text{C}$  and from 0.001 Hz to 10 Hz)

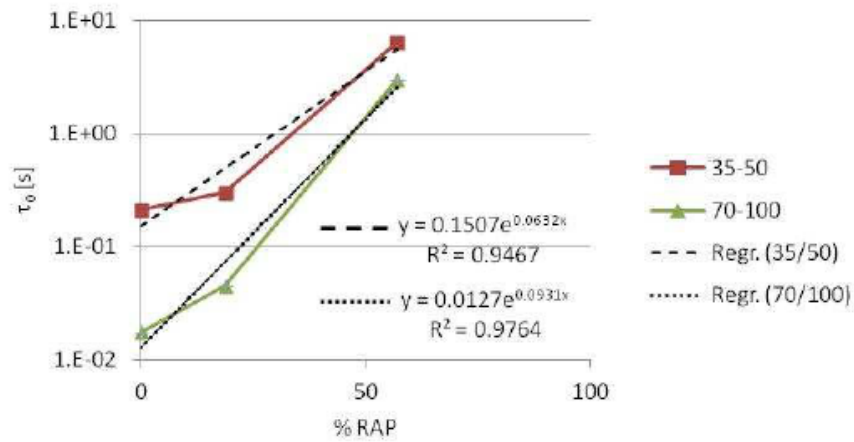
**Table 6.** 2S2P1D parameters for mixes

mix	% voids	$E_{00}$ [MPa]	$E_0$ [MPa]	k	h	$\delta$
35/50 + 0% RAP	3.74%	14	36000	0.179	0.560	2.25
35/50 + 20% RAP	4.06%	16	37400	0.173	0.540	2.40
35-50 + 60% RAP	2.34%	20	38750	0.161	0.500	2.53
70-100 + 0% RAP	3.84%	13	38600	0.179	0.540	2.15
70-100 + 20% RAP	2.61%	21	39250	0.173	0.530	2.15
70-100 + 60% RAP	2.13%	15	39800	0.160	0.500	2.50
mix	% voids	$\tau_0$ [s]	$\beta$	$v_{00}$	$v_0$	$\tau_v$ [s]
35/50 + 0% RAP	3.74%	2.10E-01	200	0.35	0.14	1.50E+00
35/50 + 20% RAP	4.06%	3.00E-01	300	0.47	0.08	1.00E+01
35-50 + 60% RAP	2.34%	6.50E+00	900	0.38	0.14	5.00E+01
70-100 + 0% RAP	3.84%	1.75E-02	400	0.51	0.20	1.00E-01
70-100 + 20% RAP	2.61%	4.50E-02	500	0.53	0.20	1.00E+00
70-100 + 60% RAP	2.13%	3.00E+00	1400	0.47	0.20	1.60E+01

Although no data is available for a mix produced only with RAP-extracted binder, 2S2P1D parameters of mixes appear to follow a tendency with RAP material content, similarly to their corresponding binders. This hypothesis will be investigated in the future. Figure 10 shows the evolution of  $\tau_0$  with RAP material content for mixes produced with 35-50 and 70-100 binders. Although observed trends are less definite than for binders, satisfactory  $R^2$  values are detected.

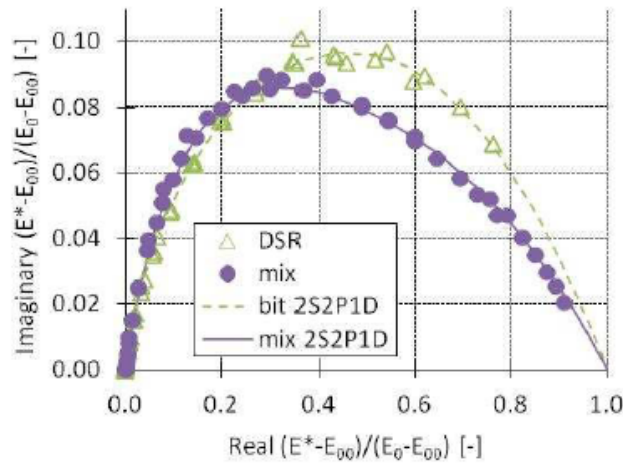


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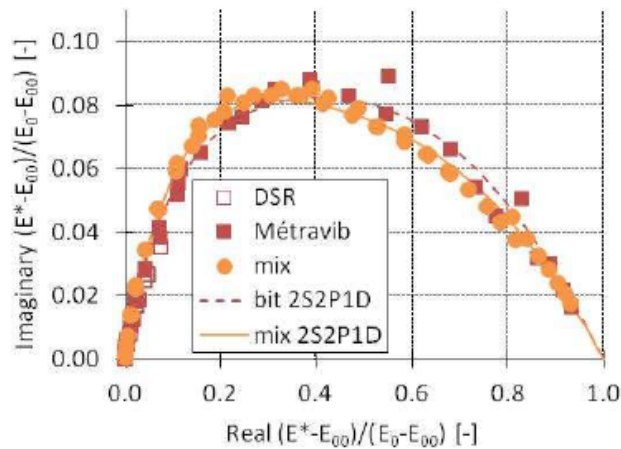


**Figure 10.**  $\tau_0$  parameter evolution with RAP material content for mixes produced with 35-50 and 70-100 base binders

Normalized complex modulus was calculated for all tested binders and mixes. Due to the more reliable results obtained, DSR+Métravib data were considered for binder blends produced with 35-50 base bitumen, while DSR data were used for binder blends produced with 70-100 base bitumen. Figures 11 and 12 show normalized Cole-Cole curves of, respectively, 70-100 + 20% RAP binder and mix and 35-50 + 20% RAP binder and mix.



**Figure 11.** Normalized Cole-Cole plot of 70-100 + 20% binder and mix (experimental results and 2S2P1D model)

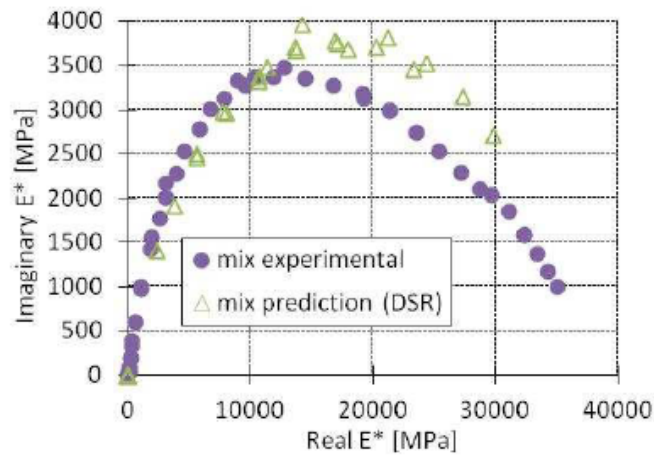


**Figure 12.** Normalized Cole-Cole plot of 35-50 + 20% binder and mix (experimental results and 2S2P1D model)

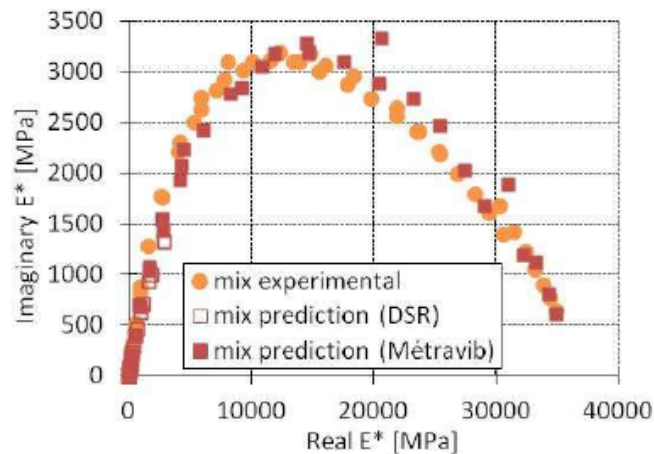
It can be noticed that normalized curves of 35-50 + 20% RAP binder and mix show a reasonable superposition, while curves of 70-100 + 20% RAP binder and mix appear to divert. The same difference has been observed for all mixes produced with 35-50 and 70-100 binders, in a consistent manner. The reason for this discrepancy is most likely to be found in the problem with the DSR measurements at low temperature.

Using normalized values of complex modulus and parameters  $E_0$  and  $E_{00}$  SHStS transformation has been applied to simulate the prediction of mix LVE behavior from the corresponding binder. Since a better correspondence with mix data was found for DSR+Métravib results than for DSR-only results, SHStS prediction for mix 35-50 + 20% RAP was applied on results for DSR+Métravib tests on the corresponding binder, while for mix 70-100 + 20% RAP DSR results were used. Figures 13 and 14 show a comparison between experimental results and SHStS predictions for, respectively, mixes 70-100 + 20% RAP and 35-50 + 20% RAP. SHStS transformation yielded successful predictions of mix LVE behavior from binder LVE behavior, when reliable input binder data were provided.

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**Figure 13.** Prediction of 70-100 + 20% RAP mix LVE behavior from DSR tests on corresponding binder using SHStS transformation

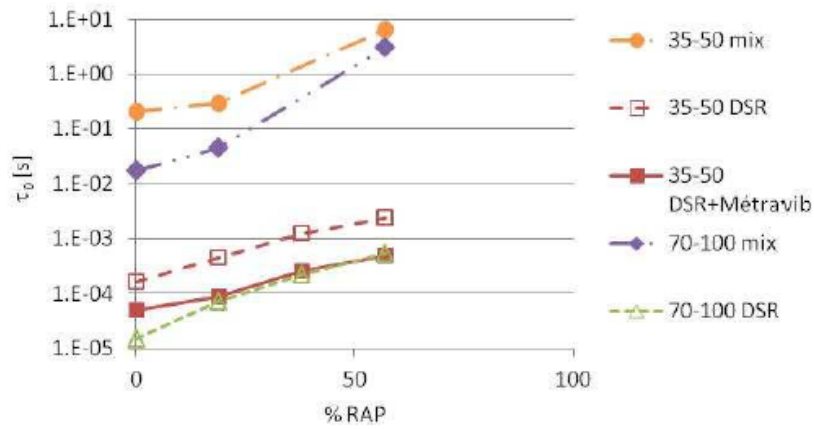


**Figure 14.** Prediction of 35-50 + 20% RAP mix LVE behavior from DSR and Métravib tests on corresponding binder using SHStS transformation

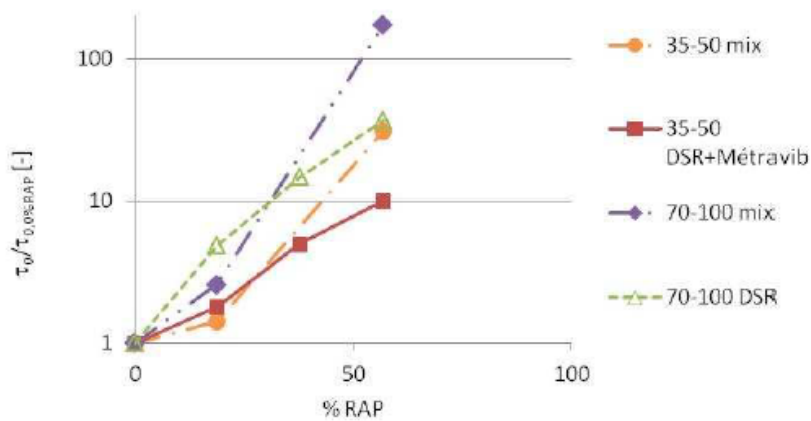
An additional comparison between mix and binder behavior was performed about the evolution of parameter  $\tau_0$  with RAP content. Figure 15 shows a summary for all materials tested in the experimental campaign (with the exception of RAP-extracted binder).

In order to immediately visualize the correspondence between related binders and mixes,  $\tau_0$  values were normalized with respect to those obtained for binders and mixes not containing RAP material (Figure 16). Results for DSR-only tests on bitumen blends produced with 35-50 binder were not taken into account.

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**Figure 15.** Comparison of  $\tau_0$  parameter evolutions with RAP material content for binders and mixes



**Figure 16.** Comparison of normalized  $\tau_0$  parameter evolutions with RAP material content for binders and mixes

Normalized  $\tau_0$  values for binders and mixes appear not to differ excessively for RAP contents up to approximately 40%. In particular, at 60% RAP content, normalized mix  $\tau_0$  values are remarkably higher than binder values. This observation proves that the diversion from a perfect linear trend of  $\tau_0$  value of mixes is caused by results obtained for the high RAP content mixes. It is the authors' hypothesis that this difference between  $\tau_0$  trends for binders and mixes is due to an incomplete blend of RAP and fresh binder within the mix. Further investigation is currently being carried out on this subject in the framework of the research project.

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## 5. Conclusions

Six asphalt mixes were produced by mixing aggregates with two straight run fresh bitumens (35-50 and 70-100 penetration grade) and a well-known RAP material. RAP material was added in the mixes in various proportions (0%, 20%, 40% and 60%). LVE properties of all mixes were measured on cylindrical samples in tension/compression mode. For all mixes, a fully blended binder was reproduced by mixing fresh and RAP-extracted binder with the same proportions. Binder blends containing 40% RAP binder were also produced using both pure bitumen bases. A total of nine binder (including RAP binder) was therefore considered in the experimental campaign. DSR (from -16.5°C to +70°C and from 0.01 Hz to 30 Hz) complex modulus tests were performed on all binders. Resulting  $E_0$  appear lower than expected. It is suspected that these values are not correct because of experimental limitations of the DSR device at low temperatures. For this reason, tension/compression tests were performed on bitumen blends of 35-50 and RAP-extracted binder using a Métravib device. Métravib data were used to replace DSR data obtained at temperatures lower than +10°C: thus, new DSR+Métravib sets of data were obtained. Experimental results obtained on these sets of data appeared more coherent with expected values and literature findings.

2S2P1D model satisfactory fitted experimental results of all binders and mixes. SHStS transformation was used to simulate the prediction of mix LVE behavior from the corresponding binder. A better correspondence with mix data was found for DSR+Métravib results than for DSR-only results. Overall, SHStS transformation yielded successful predictions of mix LVE behavior from binder LVE behavior, when reliable input binder data were provided.

The evolution of parameter  $\tau_0$  with RAP content was monitored. Concerning binders,  $\tau_0$  values resulting from DSR tests tend toward the unique value of RAP binder. The same tendency is observed for results of DSR+Métravib tests on 35-50 blends, although the corresponding fitting line does not coincide with the one obtained for DSR-only data. DSR+Métravib data show provide  $\tau_0$  values lower than DSR-only data while exhibiting an approximately parallel trend.  $\tau_0$  values of mixes resulted higher than those of corresponding binders while showing similar tendencies. Future investigation will be carried out to verify whether prediction relationships can be developed for the evolution of 2S2P1D parameters of mixes with RAP content, similarly to what already performed for bitumen blends.

In order to evaluate the correspondence between related binders and mixes,  $\tau_0$  values were normalized with respect to those obtained for binders and mixes not containing RAP material. Normalized  $\tau_0$  values for binders and mixes appear not to differ excessively for RAP contents up to approximately 40%. For high RAP contents, normalized mix  $\tau_0$  values are remarkably higher than binder values. It is the authors' hypothesis that this difference is due to a incomplete blend of RAP and fresh binder within the mix. Further investigation is currently being carried out on this subject in the framework of the research project.

## 6. Acknowledgements

The authors would like to express their deep gratitude to Stéphane Dupriet (EIFFAGE Travaux Publics) and Ronald Van Rooijen (BP: Beyond Petroleum, BP Europa SE) for their help and effort in the experimental campaign and to the laboratories of EIFFAGE Travaux Publics (Corbas and Ciry Salsogne, France) and BP Europa SE (Gelsenkirchen, Germany).

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# Annexe 5

## Comportement des matériaux composites traités au liant hydraulique et incluant des fraisâts bitumineux (2008-2012)

L'article de congrès international suivant est fourni en illustration de ladite recherche:

- K. Bilodeau, C. Sauzéat, H. Di Benedetto, F. Olard, Roller compacted concrete for road base layer with RAP and steel fibers: viscous properties and description of experimental site, 10<sup>th</sup> International Conference on Concrete Pavements, Quebec City, 2012.



## Roller Compacted Concrete for road base layer with RAP and steel fibers: viscous properties and description of experimental sites

Bilodeau, K.<sup>1</sup>, Sauzéat, C.<sup>2</sup>, Di Benedetto, H.<sup>3</sup>, Olard, F.<sup>4</sup>

### Abstract

A high-performance composite material consisting of reinforced compacted cement concrete including Reclaimed Asphalt Pavement (RAP) has been investigated. This composite material is used as a base layer in combination with an asphalt overlay for heavy traffic roads. Steel fibers are used to minimize crack opening. RAP is added to preserve raw material and to comply with sustainable development, especially in the case of in-situ treatments. Composites with different RAP and fiber content and cement content were studied both in laboratory and on site. The laboratory study is divided into two parts. First, classical standard tests for road concrete were carried out: compressive strength, compressive modulus and tensile splitting strength, with a view to the design of pavement structures to be used on site. Secondly, more comprehensive rheological tests, such as complex modulus and fatigue tests, were performed to obtain a better knowledge of material behavior. Then, on-site behavior was studied from two experimental sites. The presence of bitumen in cement-treated composites induces an increase of viscous properties, observed mainly on the phase angle. These composites respect the time-temperature superposition principle and can be fitted by a visco-elastic rheological model.

### Introduction

This research is part of a French National Research Agency (ANR) project named "Recyroute". This 2.3 million euro project intends to propose a new high performance and long-lasting pavement using steel Fiber reinforced Roller Compacted Concrete (FRCC) as a road base layer including Reclaimed Asphalt

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Pavement (RAP). Pavement design, characterization of material properties and road structure calculation are part of the developments to be conducted among the involved teams: Eiffage Travaux Public, University of Lyon/École Nationale des Travaux Publics de l'État (ENTPE), Institut Français des Sciences et Technologies des Transports, de l'Aménagement et des Réseaux (IFSTTAR), Laboratoire Régional de l'Ouest Parisien (LROP), Chaussée Technique et Innovation (CTI), Paris City and Autoroutes Paris Rhin Rhône (toll highway company, EIFFAGE subsidiary). Both in situ and laboratory studies are performed. The new aspect introduced in the program is the analysis of the use of different RAP contents (between 0% and 80% of the aggregate mass).

The first step of the program is the characterization of the selected mixes to be used on experimental sites by a laboratory study divided into two parts. The first part of the laboratory study aims at determining the classical mechanical properties. They were obtained from standard tests in order to define parameters of the composite materials to be placed as a road base layer. The obtained parameters were used for pavement design. Then, a more comprehensive thermo mechanical test (complex modulus) is performed to determine the viscous properties of mixes and the effect of RAP content in mixes. The second step of this project is the monitoring of two experimental sites. The following sections describe the tested materials, the testing device and the visco-elastic rheological model considered. The rheological model use to analyse experimental data is base on simple combination of physical elements. It is composed of two Springs, two Parabolic elements and 1 Dashpot (2S2P1D) (Di Benedetto et al. 2004 & Olard et al. 2003). This paper also presents visco-elastic properties of cement-treated materials with different RAP contents and a description of the two experimental sites.

### Materials

FRCC (Ficherouille and Henin 2004) components are presented in this section, as well as material design. The hydraulic binder content is fixed at 12% by mass of the granular part. It corresponds to about 280kg/m<sup>3</sup> of concrete when used with natural aggregates only. The chosen hydraulic binder (Ligex FPL2, Calcia) is composed of 52% clinker, 21% fly ash, 6% slag and 21% limestone. The plasticizer admixture content is fixed at 0.5% in mass of hydraulic binder. Different fiber contents from 0 to 25 kg/m<sup>3</sup> are investigated. The chosen steel fibers are 6cm long and have a 0.75mm diameter (Dramix 80/60 BN from Beckaert Company, Figure 1). Fibers are added during the final mix process and are not considered for the concrete design.



Figure 1 Dramix 80/60 BN fibers used in FRCC materials

As mentioned previously, the innovative aspect is the use of RAP as an aggregate. Three RAP contents materials are studied. The contents are fixed at 0%, 40% and 80% by weight of the aggregates (respectively 0%, 36% and 70% of aggregates + hydraulic binder mass). The obtained materials are named F0%, F40% and F80%. Considering the combination of fiber and RAP contents, five different FRCC mixes are studied.

Another well known material developed by Eiffage Travaux Publics is used as reference to evaluate the properties of FRCC materials. This reference material is composed of reclaimed asphalt pavement treated with a road hydraulic binder (ERTALH®). It is made with the same Ligex FPL2 hydraulic binder (the content of which being fixed at 5% by weight of the aggregates) and contains 70% of the same RAP (by mass of aggregates + hydraulic binder).

Table 1 presents the properties of the RAP used. RAP bitumen content was obtained following the standard NF EN 12697-1. A penetration test, as well as a ring and ball test were done according to standards NF EN 1426 and NF EN 1427 on the bitumen extracted by distillation (NF EN 12697-3). Grading curves of all used aggregates (alluvial sand 0/4 Gurgy, crushed sand 0/6 Crain, crushed gravels 6/14 Crain and RAP) are presented in Figure 2a.

The final mix design was obtained with the following procedure. First the cement content and the grading envelope (Figure 2b and 2c) were fixed. Cement content was fixed in dry mass at 12% for FRCC and at 5% for ERTALH®. Then, the aggregate fractions were adjusted to obtain a grading curve inside the grading envelope specification. Figure 2b and 2c show the selected grading envelopes and grading curves of all mixes. The water content was then determined from the modified Proctor test NF EN 13286-2 and the immediate bearing ratio (IBR) test (NF P 94-078) on materials without steel fibers. Table 2 gives the mix proportions (without fibers) of the tested materials.

Table 1 Properties Of Used Reclaimed Asphalt Pavement

RAP	Aggregates	Bitumen		
		Content (%) NF EN 12697-3	Penetration at 25°C (1/10 mm) NF EN 1426	Ring & Ball temperature (°C) NF EN 1427
Yonne	0-14	3.51	12	69.4

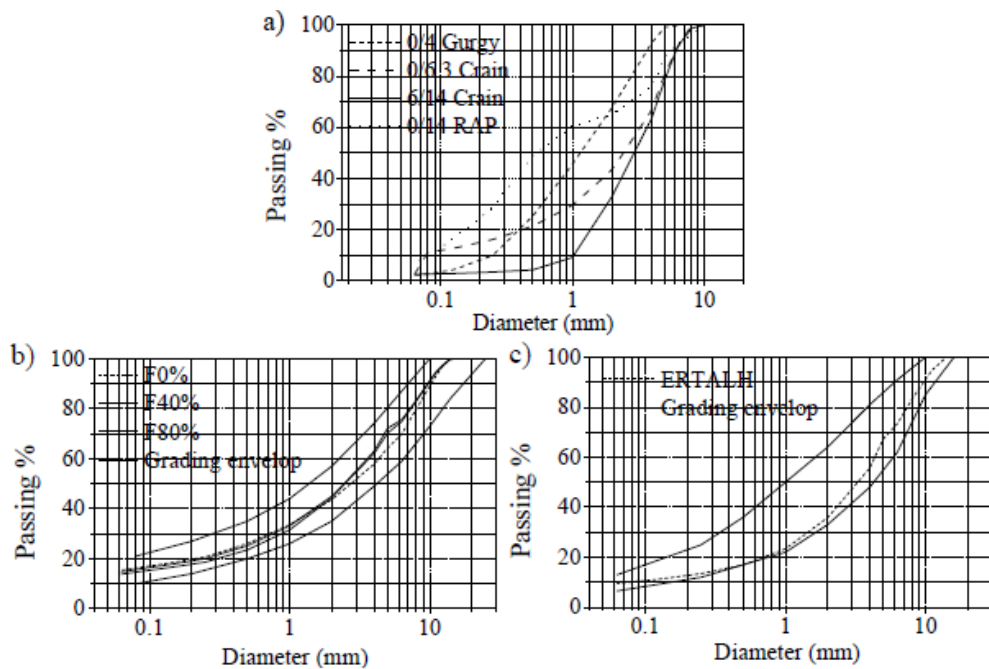


Figure 2 Grading curves and grading envelopes: a) All granular fractions; b) FRCC; c) ERTALH

Table 2 Mixes Component (Without Fibers)

Material or Properties	F0%		F40%	F80%		ERTALH		Aggregate mass
	-	-	RAP	RAP	RAP	RAP	RAP	
Type of RAP	AG	AG	Yonne	Yonne	Tours	Yonne	Tours	
Type of aggregate	Crain	Ht-Lien	-	-	-	-	-	
Alluvial sand 0/4	23%	20%	20%	18%	18%	-	-	
Crushed sand 0/6.3	35%	37%	20%	-	-	25.0%	25%	
Crushed gravel 6/14	30%	31%	12%	-	-	-	-	
RAP (0/10)	-	-	36%	70%	70%	70%	70%	
Hydraulic binder (FPL2)	12%	12%	12%	12%	12%	5%	5%	
Total (aggregates + hydraulic binder)	100%	100%	100%	100%	100%	100%	100%	
Additive <sup>1</sup> (Sika)	0.06%	0.06%	0.06%	0.06%	0.06%	None	None	
Water content <sup>2</sup>	6.1%	5.5%	6.2%	6.2%	6.1%	6.9%	6.1%	
Bitumen content (%) from RAP	0.0	0.0	1.26	2.46	2.681	2.46	2.681	
Dry Density	2.240	2.320	2.190	2.128	2.210	2.105	2.190	
Immediate bearing ratio	215	310	120	101	91	84	65	
water/cement ratio	0.508	0.458	0.516	0.516	0.508	1.380	1.220	

<sup>1</sup> by weight of concrete (Total + water)

<sup>2</sup> by weight of Total (aggregates + hydraulic binder)

### Testing devices

Complex modulus of mixes was measured with tension/compression tests. Binder complex modulus tests were carried out using the Dynamic Shear Rheometer (DSR) device. Complex modulus tests were performed on FRCC & ERTALH containing RAP and on the bitumen extracted from these RAP. Complex modulus was used to determine the influence of bitumen on the behavior of concrete materials containing RAP.

Visco-elastic properties of mixes were obtained by tension/compression complex modulus tests. The principle of this test consists in applying a sinusoidal loading on a cylindrical specimen of cement-treated materials. The tested specimens were 120 mm high, with a 75 mm in diameter. During the test, axial and radial strains as well as axial stress (radial stress is nil) were registered. Axial stress ( $\sigma_1$ ) was measured from a load cell, while axial strains ( $\epsilon_1$ ) were obtained with three extensometers. Radial strain ( $\epsilon_2$ ) was deduced from two non-contact transducers (Figure 3). For the complex modulus test, cement-treated materials were loaded at 7 different frequencies (from 0.01 to 10 Hz) and 8 temperatures (from -15°C to +45°C). The target axial strain was fixed to 50 $\mu$ def.

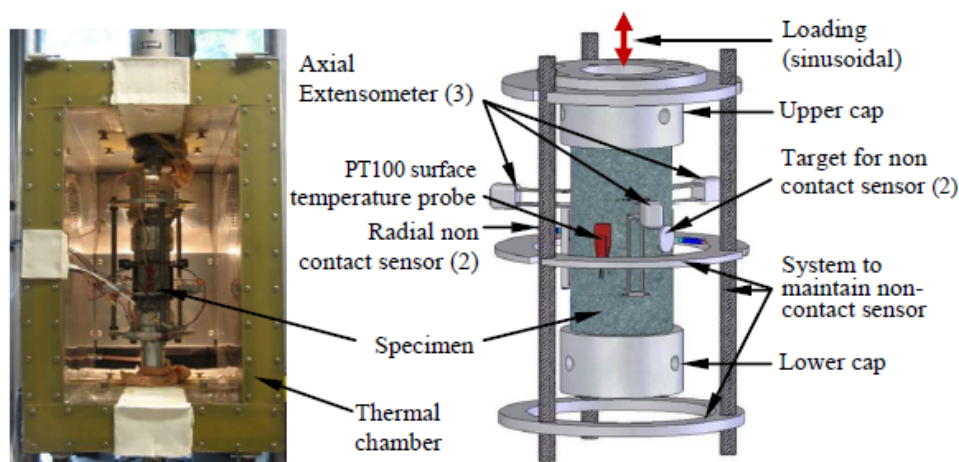


Figure 3 Experimental device used for tension-compression test, in the linear domain.

### 2S2P1D visco-elastic rheological model

Before showing the results and the analysis, the authors present an alternative general rheological model 2S2P1D, generalization of the Huet-Sayegh model, valid for bituminous binders and mixtures. This general model is based on a simple combination of physical elements (two springs, one dashpot and two parabolic elements) (Figure 4).



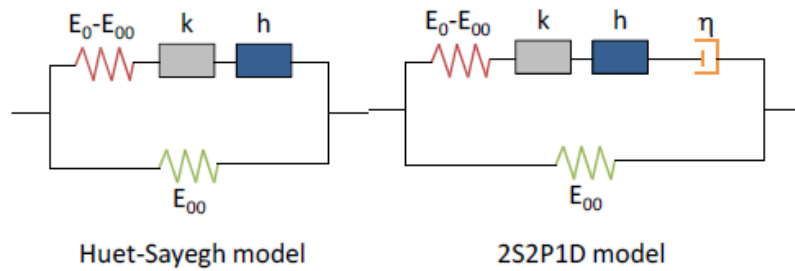


Figure 4 Representation of the Huet-Sayegh and general 2S2P1D for bituminous binders and mixes, h and k refer to two parabolic creep elements.

The Huet-Sayegh model (Sayegh 1965) has been adapted by adding a linear dashpot in series with the two parabolic elements and the spring of rigidity  $E_0-E_{00}$  (Figure 4). Thus, in particular for binders, for which  $E_0=0$ , the model is equivalent to a linear dashpot at very low frequencies. At a given temperature, the 2S2P1D model has 7 constants and the complex modulus is given by the following expression:

$$E_{2S2P1D}^*(\omega) = E_{00} + \frac{E_0 - E_{00}}{1 + \delta(i\omega\tau_E)^{-k} + (i\omega\tau_E)^{-h} + (i\omega\beta\tau_E)^{-1}} \quad (1)$$

with  $0 < k < h < 1$  and with a Newtonian viscosity  $\eta = (E_{00} - E_0)\beta\tau$ . When  $\omega\tau \rightarrow 0$ , then  $E^*(i\omega\tau) \rightarrow E_0 + i\omega(E_{00} - E_0)\beta\tau$ .  $\beta$  is dimensionless.

It has to be emphasized that only 7 constants ( $\delta$ ,  $k$ ,  $h$ ,  $E_{00}$ ,  $E_0$ ,  $\beta$  and  $\tau_0$ ) are needed to entirely determine the linear viscoelastic behavior of the considered material, at a given temperature. For binders, the experimental static modulus  $E_{00}$  is very close to zero. Thus, in equation 1,  $E_{00}$  can be taken equal to zero for binders and the number of constants of the model can be reduced to six.

Furthermore, if the hypothesis of a linear viscoelastic thermorheologically simple behavior can be applied to the considered material (i.e. the Time-Temperature Superposition Principle (TTSP) holds), only the  $\tau$  parameter depends on temperature.  $\tau$  evolution may be approximated by a WLF type law (Williams et al. 1955) (equation 2) in the range of temperatures observed in laboratory (Olard et al. 2003 & Neifar et al. 2001). If TTSP holds, the two additional constants  $C_1$  and  $C_2$  (calculated at the reference temperature  $T_{ref}$ ) of the WLF equation are needed. The number of constants of the model amounts to 9 (8 for binders, since  $E_{00}=0$ ). Unlike the Huet model and like the Huet-Sayegh model, no analytical expression of the creep function of this model is available in the time domain. Some simulations are shown in the next paragraph.

$$\log(a_T) = -\frac{C_1(T - T_{ref})}{C_2 + T - T_{ref}} \quad (2)$$

### Visco-Elastic properties

Visco-elastic properties of cement-treated materials containing reclaimed asphalt pavement were obtained from complex modulus tests under tensile-compressive cyclic loading. Experimental results of complex modulus tests on the studied composite materials are presented in Figures 5 and 6. The 2S2P1D rheological model is also fitted to experimental data. Figure 5 presents complex modulus results in Cole-Cole and Black representations. Each group of materials (ERTALH, F80% and F40%) is plotted separately.

Results for F80% materials with the same range of curing time but with different fiber contents are similar to each other (same range of stiffness modulus and phase angle). Furthermore increasing the cement content between ERTALH and F80% causes a higher complex modulus without significant changes for the phase angle. The effect of curing time (60 days and 772 days) was studied for F40%. The phase angle of mixes with fibers decreases when curing time increases. Results presented in Black space shows that RAP content influences viscous properties. As observed by some authors (Bilodeau et al. 2011, Mathias 2004, Delwar et al. 1997 & Kolias 1996 (1&2)), complex modulus decreases as RAP content increases. Results for phase angles show small viscous properties. The studied composites materials have a unique curve in Black, therefore the hypothesis of TTSP is hold.

For each studied material, master curves of the norm of complex modulus and the phase angle are plotted (Figure 8) at a reference temperature of 15°C. Adding RAP to FRCC decreases the norm of the complex stiffness modulus ( $|E^*|$ ) and increases the phase angle. At same RAP content (80%) and with lower cement content (F80% and ERTALH) the norm of complex modulus decreases. Compared to the behavior generally observed on bituminous materials (Olard 2003), the maximum phase angle of cement-treated materials including RAP is very low. It reaches about 3°-7°. But curves obtained for mixes and the bitumen have the same shape regardless of the representation. ERTALH materials tested at different curing times show similar results, such as the minimum asymptotic  $E_{00}$  modulus ranging between 4500 and 5500 MPa. The same tendency is observed on the maximum asymptotic  $E_0$  modulus (14000 MPa). The maximum and minimum ratio ( $E_0/E_{00}$ ) of composites tested is about 1.5 to 3, depending on the considered mixes. This value is very low compared to asphalt concrete mixes for which obtained  $E_0/E_{00}$  is about 1000 (Olard 2003).

Simulation curves with the 2S2P1D rheological model are also plotted with our experimental results. These simulations are very close to the experimental results. 2S2P1D parameters used for each mixes are reported in Table 3.

Figure 7 presents shift factors as a function of temperature. They were used to build the master curve of all materials. These experimental values are fitted with a unique set of the classical Williams-Landel-Ferry (WLF) law parameters ( $C_1$ ,  $C_2$ ) for the chosen reference temperature. Shift factors used for all mixes containing RAP are very close to those for the bitumen.

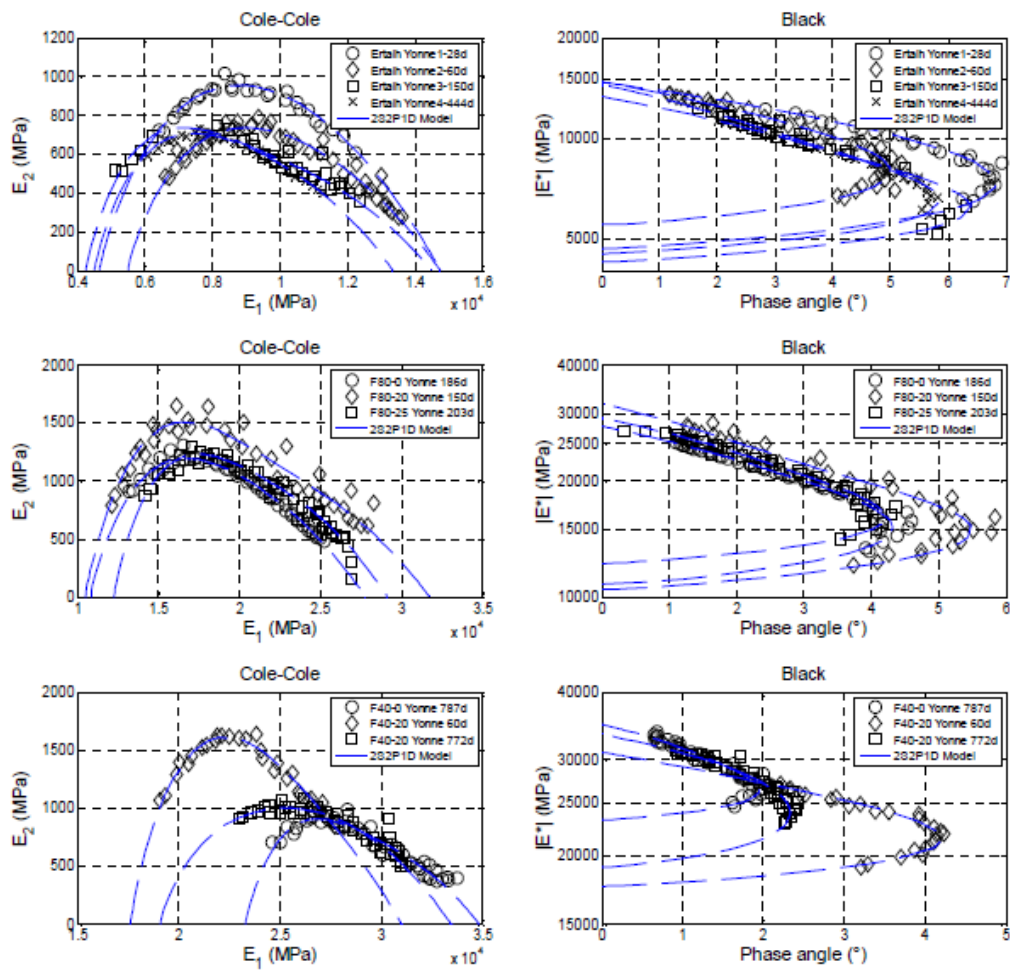


Figure 5 Complex modulus results: a) Cole-Cole; b) Black space

Table 3 2S1P1D parameters of all mixes at  $T_{ref} = 15^{\circ}C$ .

Mixes	$E_{00}$	$E_0$	$k$	$h$	$\delta$	$\beta$	$\tau$
Ertalh 28d	4650	14750	0.195	0.45	1.95	2000	47.3
Ertalh 60d	5500	14750	0.156	0.4	1.8	2000	230.7
Ertalh 150d	4250	14500	0.13	0.45	1.95	2000	144.7
Ertalh 441d	4500	13350	0.153	0.45	1.9	2000	243.3
F80-0 186d	10850	27800	0.13	0.37	1.7	2000	409.0
F80-20 150d	10500	31750	0.118	0.41	1.7	2000	20.0
F80-25 203d	12250	29150	0.135	0.41	1.9	2000	64.1
F40-0 787d	23300	34800	0.141	0.45	1.8	2000	325.5
F40-20 60d	17600	31000	0.193	0.55	1.3	2000	22.3
F40-20 772d	19100	33450	0.16	0.4	2.9	2000	10090.0

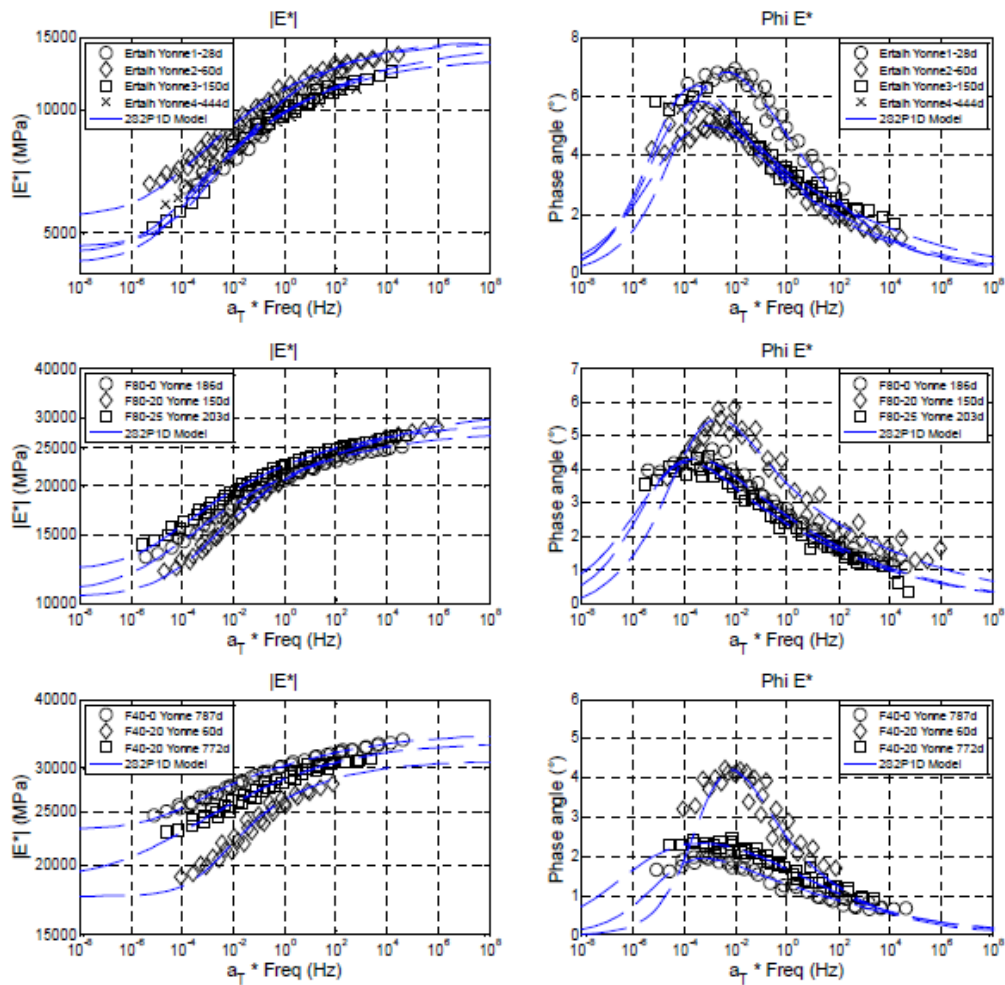


Figure 6 Master curves for all materials ( $T_{ref} = 15^{\circ}C$ ) : left) Norm of complex modulus; right) Phase angle

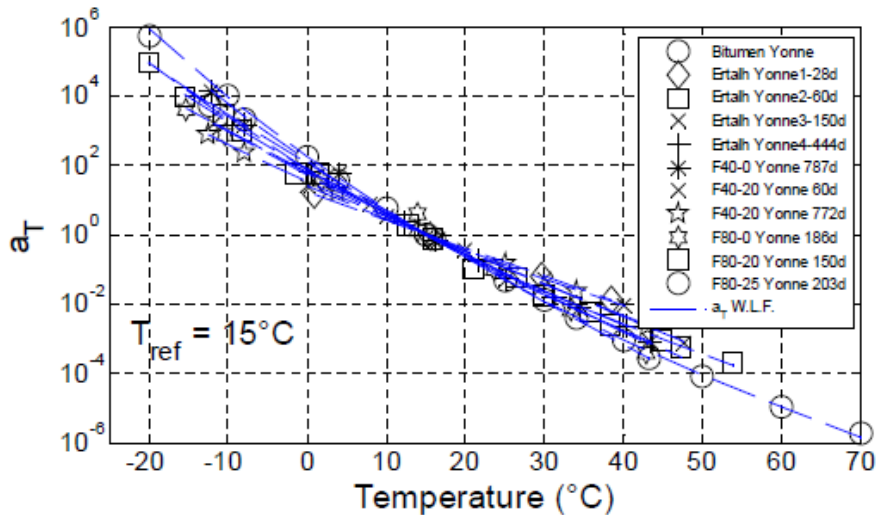


Figure 7 Shift factors used to translate isotherms of mixes and RAP-extracted bitumen.

**Description of the two experimental sites**

**A6 Venoy Service Area.** This experimental site is a rehabilitation of a part of the Venoy truck service area on highway A6 near Auxerre, France. Figure 8a shows the details of this experimental site finished in October 2009. The site is divided in two lanes and four sections. Trucks come from Lyon and enter by the access lane (Figure 8b). They leave the site through the exit lanes after passing the parking zone. The selected mixes to be placed on site are F0% (30), F40% (25), F80% (25) and ERTALH<sup>®</sup>. Due to actual limitation of fiber and cement materials used, the fiber content of FRCC<sup>™</sup> with RAP is higher than initially planned and is fixed to 25kg/m<sup>3</sup>.

First operations on site are the milling of current asphalt layer to the desired thickness and off-site storage. The RAP mills from this operation are not the RAP used in our project. Ours RAP is already recrushed and sieved to create a new RAP aggregate (0-14 mm). Afterwards a thin layer of asphalt emulsion (800g/m<sup>2</sup>) with gravel is applied between the existing layer after milling and the new base layer (ERTALH<sup>®</sup> or FRCC<sup>™</sup>). Base layer mixes are produced from a mobile plant (Figure 8c) installed near (about 25km) the experimental site. They are placed by means of paver that is similar to the one used for asphalt concrete or a roller-compacted concrete. Afterwards the targeted density is obtained with both vibratory asphalt compactor and a pneumatic tire compactor. At last another asphalt emulsion layer (400g/m<sup>2</sup>) with gravel is applied before surfacing with asphalt concrete.

During construction, strain gages were placed in the longitudinal and transverse directions below ERTALH<sup>®</sup> and FRCC to monitor strain (Figure 8d) under static and mobile loads. Measurements with these gages and visual characterization of this high potential composite material will be realized at specific times during 3 years.

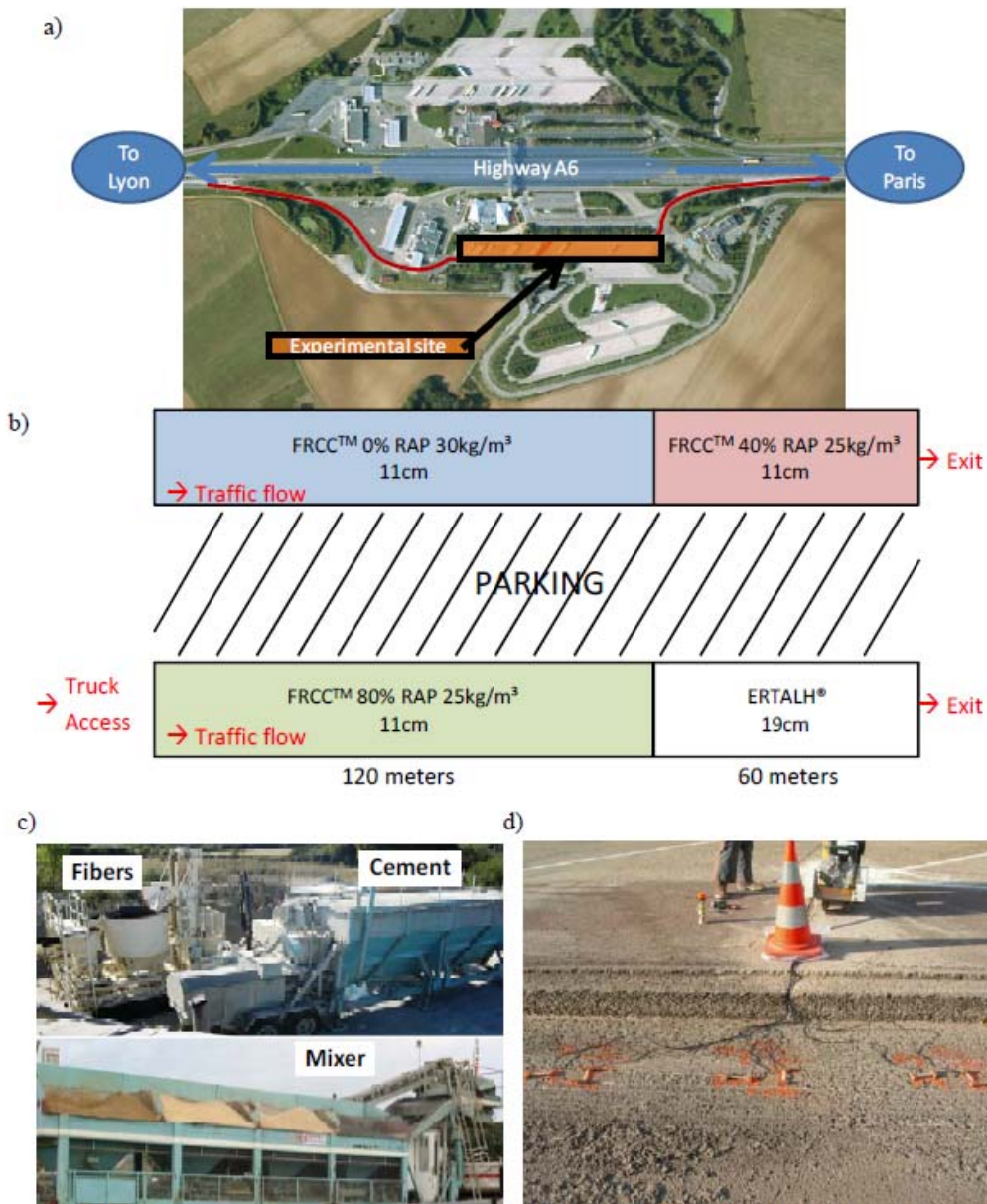


Figure 8 Description of on-site experimentation: a) Aerial view of the site; b) Top view of the site; c) View of mobile plan; d) Strain gages below the base layer

**IFSTTAR'S accelerated pavement testing facility.** The IFSTTAR's Accelerated Load Testing (ALT) facility, in Nantes, is an outdoor circular carousel dedicated to full-scale pavement experiments, carried out with public and/or private partners. The carousel is composed of a central tower and four arms (20 m long each) equipped with wheels, running on a circular test track. The experimental circular pavement has an average radius of 17.5 m and a width of 6 m, thus a total length of approximately

110 m (Kerzreho et al. 2008). The position of the loading module can be modified and adjusted at a different radius on each arm. The effect of different loading configurations can be observed simultaneously. During loading, a lateral offset of the load can be applied to simulate the lateral distribution of loads of real traffic (Autret 1988).

The arms can be equipped with various load configurations: single or dual wheel; single, tandem or triple axles. A large range of loads can be applied: from 45 to 80 kN on a single half axle, up to 135 kN with multiple axles. The maximum speed in classical load configuration (65kN twin-wheels) is 100 km/h. A low-stiffness suspension system allows controlling the loads applied to the experimental pavement during the entire test, regardless of the degradation of the pavement is. Pavement structures are built by private construction firms, using ordinary road building equipment.

*Concept of experimental pavement structures.* Eight different structures were tested simultaneously, on two different radiuses (respectively 16 m and 19 m, with 4 structures each) of 3m of width. This experiment focuses mainly on the evaluation of new pavement structures produced with new base layer materials. Tested structures include: one reference structure made with a French standard high modulus bituminous material (EME2) with a well known behavior; one structure made of RAP aggregates treated with hydraulic binder (ERTALH<sup>®</sup>); six structures using FRCC materials; three including RAP and three without RAP. Two kinds of new aggregates are used for structures without RAP: the “Crain” (soft limestone) and the “Haut-Lieu” (hard limestone).

These structures were preliminarily designed using the French official design method. Some additional design parameters were taken into account for the innovative structures not considered by the official method. Final structures and carousel compositions are described in Figure 8.

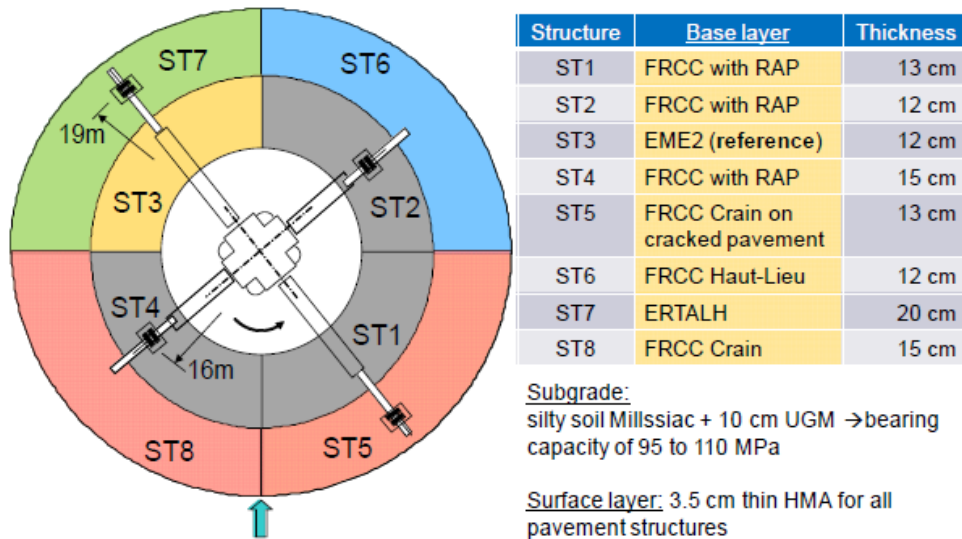


Figure 8 Schematic of tested structures at the IFSTTAR's ALT facility.

For the entire carousel, the same subgrade soil and unbound granular material is used as a support layer. Materials used for base layers and surface layers are described below. As shown in Figure 8, five different materials were used for the base layer of the eight experimental pavement structures. Except for the high modulus asphalt material EME2, which was produced and transported from a bituminous material production plant, all other materials were produced by an on-site production plant. Two kinds of bituminous materials were used as a surface layer for these pavement structures:

- a 0/10 HMA with a 35/50 penetration grade ordinary bituminous binder is used on all inner radius structures and on structure 7 (outer radius);
- on structures 5, 6 and 8, on the outer radius, the surface layer consisted of a 0/10 HMA with an SBS-modified binder, called Orthoprene®, developed by EIFFAGE.

A total traffic of more than 2.15 million heavy load passages was applied over nine months. It should be noted that the loading started three months after construction of the pavement structures, so that the cementitious materials used in seven innovative structures reached their full resistance. In addition, the first 100,000 loads were carried out with the dual wheels of 45 kN (50,000 loads) and of 55 kN (50,000 loads). Later, the reference French standard dual wheel of 65 kN was used until 2 million passes. At the end of the experiment, about 21,000 and 150,000 additional dual wheel load passings of respectively 70 kN and 75 kN, were applied to study the overload effect on the tested pavement structures. According to the French rational design method, the aggressiveness of the loading on each structure depends on the slope of the fatigue law of the material used in the base layer, as follows:



$$NE = NP \left( \frac{P}{P_{ref}} \right)^{-1/b} \quad \text{with } P_{ref} = 65 \text{ kN} \quad (3)$$

Where NP is the number of passes of the load P and NE is the number of reference loads (65kN) equivalent to the NP passes of the load P. Table 4 shows the cumulated equivalent traffic (standard load of 65 kN) for each tested structure corresponding to a given base layer material, according to equation 3.

Table 4 Cumulated equivalent traffic for each tested structure

Structure	S2-S1-S4	S3	S5-S8	S6	S7
Material	FRCC with RAP	EME2	FRCC Crain	FRCC Ht-Lieu	ERTALH
-1/b	9.9 (*)	6.6 (**)	16 (*)	16 (*)	10.7 (*)
NE (65 kN)	2.6 x 10 <sup>6</sup>	2.4 x 10 <sup>6</sup>	3.5 x 10 <sup>6</sup>	3.5 x 10 <sup>6</sup>	2.7 x 10 <sup>6</sup>

(\*) slope of the fatigue law to preliminary design of pavements  
 (\*\*) slope of the fatigue law according to fatigue tests on EME2

For almost all applied loads, the rotational speed is 10 rotations/min, which corresponds to a linear speed of 71.6 and 60.3 km/h for the radius of 19m and 16m respectively. Moreover, during a specific measurement (deformation and dynamic deflection measurements), the rotational speed of the carousel can be set at different values, from 0.6 to 10 rotations/min. Lateral offset of the loads is also applied during the experiment. For this experiment, the wheel path resulted of a lateral range of 1.6m wide, distributed according to an approximated normal law.

In each structure, ten horizontal strain sensors were placed at the bottom of the base layer for the measurement of longitudinal (5 sensors) and transversal (5 sensors) reversible strains. These sensors were positioned in the middle of the outer and the inner wheel-paths. In the middle of each structure, an anchored deflectometer was placed inside the pavement for dynamic deflection measurement. Thermocouples were also placed inside three pavement structures made of bituminous material (ST3 – EME2) or including RAP (ST4 – FRCC with RAP, ST7 – ERTALH) at different depths in order to monitor the temperature in each layer of these structures. Two other thermocouples were also used to measure air temperature in sunlight and in shade. Measurements of deflections and transversal profiles were also performed after different levels of traffic, and visual surveys of the surface of the pavement were carried out every day.

### Conclusions

From the presented experimental study, the following conclusions can be drawn:

Cement-based materials produced with the addition of RAP material, such as materials, showed light viscous properties. A unique normalized Black (or Cole-Cole) curve can be obtained for materials having very different stiffness, provided

they contain the same bitumen, even in very small quantity. TTSP was verified for these materials.

Master curves were built for complex modulus. Shift factors used are close, showing that a unique set of shift factors, in first approximation, describes the behavior of the material with respect to the Time-Temperature Superposition Principle. Shift factors are adequately fitted by the Williams-Landel-Ferry (WLF) law (eq. 2). WLF curves for bitumen and FRCC or ERTALH mix produced from the same binder were almost coincident with the WLF curve of the binder itself. The temperature dependency of a bituminous material is driven by the binder, regardless of the aggregates skeleton.

Two experimental sites were described in this paper. The first one, a rehabilitation of A6 Venoy serviced area. It shows the possibility to use these kinds of composites as pavement structure reinforcement. The second one is an accelerated pavement testing to the IFSTTAR alt Facility. First results show that all pavement structure had resisted to all. Further investigation of these two experimental sites will provided conclusions about pavement design parameters to be used for these composites.

#### **Acknowledgement**

Authors would like to thank Jean-Maurice BALAY, Jean-Pierre KERZREHO and Mai-Lan NGUYEN from IFSTAR to their contribution to this research and particularly for the IFSTTAR'S accelerated pavement testing facility section.

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## Annexe 6

### Procédés d'enrobage à température réduite. Evaluations mécanique et environnementale (2005-2012)

Deux articles de rang A sont fournis pour illustration des travaux concernant les enrobés semi-tièdes:

- A. Romier, M. Audeon, J. David, Y. Martineau, F. Olard, Low-Energy Asphalt with Performance of Hot-Mix Asphalt, Transportation Research Record: Journal of the Transportation Research Board, No. 1962, Transportation Research Board of the National Academies, Washington D.C., pp. 101-112, 2006.
- V. Gaudefroy, F. Olard, B. Cazacliu, C. de La Roche, E. Beduneau, JP. Antoine, Laboratory investigations of mechanical performance of foamed bitumen mixes that use half-warm aggregates, Transportation Research Record: Journal of the Transportation Research Board, No. 1998, Transportation Research Board of the National Academies, Washington DC, pp. 89-95, 2007.

Les deux brevets européens EP1717369 et EP1712680 sont par ailleurs fournis.



# Low-Energy Asphalt with Performance of Hot-Mix Asphalt

Alain Romier, Maurice Audeon, Jac David, Yves Martineau, and François Olard

The paper presents an original process to produce a warm asphalt mixture: low-energy asphalt (LEA) with the performance of hot-mix asphalt (HMA). The Kyoto agreements continue to divide the planet, while protocol signatories are defining their application conditions. People wonder what the resulting repercussions will be on their respective fields of activity and their quality of life. A new aggregate coating technique is being presented in that context. The technique, the LEA process, substantially reduces energy consumption and greenhouse gas emissions. The low-energy coating technique LEA (EBE, from the French “enrobage à basse énergie” involves the manufacture and application of asphalt mixes at a temperature lower than 100°C. These mixes offer a performance equivalent to that of HMA. They combine the action of the mixing energy, temperature, and water on the components of the mix, its bitumen, and aggregate skeleton. The originality of the process lies in the best use of changes in the condition of the bitumen: fluid when it is hot and able to transform into foam or to emulsify when in contact with water. Moreover the moderate-temperature heating of only the chippings (coarse aggregates) is involved; the rest of the aggregate skeleton is used cold and wet. This technique yields a significant saving in mixing energy and reduces the resulting gas emissions. The paper looks briefly into the process and sets it within its appropriate context.

The Kyoto agreements continue to divide the planet, while protocol signatories are defining their application conditions. People wonder what the resulting repercussions will be on their respective fields of activity and on their quality of life. That is the context in which a new aggregate coating technique is being presented. The technique, the low-energy asphalt (LEA) process, substantially reduces energy consumption and greenhouse gas emissions.

The LEA technique (known in French as EBE, for *enrobage à basse énergie*) involves the manufacture and application of asphalt mixes at a temperature below 100°C. These mixes offer performance equivalent to that of hot-mix asphalt (HMA). They combine the action of the mixing energy, temperature, and water on the components of the mix, its bitumen, and aggregate skeleton. The originality of the process lies in the best use of changes in the condition of the bitumen: fluid when it is hot and able to transform into foam or emulsion when in contact with water. The process moreover involves the moderate-

temperature heating of only the coarse aggregates; the rest of the aggregate skeleton is used cold and wet. The technique yields a significant saving in mixing energy and reduces the resulting gas emissions (1–3). This paper looks briefly at the process and sets it within its appropriate context.

## BACKGROUND AND POSITIONING

The road industry has for many years been seeking to minimize the amount of energy required to manufacture HMA and to lower asphalt plant emissions, combining energy savings and environmental benefits.

Extensive ongoing research attests to the determination to find solutions, in particular in the area of warm mixes (WMA). Some do offer moderate reduction in manufacturing temperature (20°C to 40°C) and exhibit properties matching closely those of hot-mix asphalt. But they rely mostly on binders or combinations of binders or combinations of binders and additives modifying the properties of asphalt during mix manufacturing and laying down. Few of these new processes have made any tangible strides.

Many recent international technical articles and papers have described and provided overviews of available techniques and their uses (4–11). Temperature reductions on the order of 20°C to 40°C have been obtained.

LEA is substantially different. It relies on the use of a much lower temperature than that of HMA or WMA. LEA mixes are produced at a temperature below 100°C and laid down at 90°C to 60°C.

## GENESIS OF PROJECT

2002. Basic principles of the technique and its applications were formulated.

2003. French, European, and international patents were filed. Financial support was secured from a French R&D agency (ANVAR). First, LEA mixes were produced and tested in a government laboratory (LROP). Technical studies were pursued, in particular in the thermal and environmental areas. The first street paving was done with an industrial prototype batch plant in November 2003.

2004. Extensive R&D program was carried out at a French Advanced School of Engineering (ENSAM) involving the Thermodynamics Laboratory of Professor Yves Le Goff and Microstructures Laboratory of Nicolas Hueber. Further tests were carried out in government laboratories.

2005. LEA reached the industrial stage with the customization of LEA mix production by an AMMANN batch plant near Paris. The first industrial manufacturing operations began in April 2005.

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### LEA ESSENTIAL INNOVATION

The LEA process differs substantially from hot- and warm-mix asphalt processes. It is based on a global reassessment of the basic thermodynamics of HMA production to minimize or eliminate the most energy-consuming/pollution-producing phases.

Some of the observations that led to LEA development follow:

- In a traditional HMA process a great deal of energy is spent drying aggregates, that is, evaporating water (the latent heat of evaporation of water represents five times the energy required to raise the same mass of water from 0°C to 100°C). The energy required for heating dry, coarse aggregates at the coating temperature is five times lower than that used to raise the same temperature of an equivalent mass of water. Exceeding the 100°C threshold greatly increases the heating energy expenditures.
- Aggregate skeleton heating conditions during hot mixing will influence energy consumption.
- Fine aggregates (sand and filler) have the highest moisture content (i.e., in France the water content in 0 to 2 mm sand, and 18% passing 80 micrometer sieve, varies between 2.5% and 4.7% between the winter and summer periods) and require proportionally much more drying energy than do coarse aggregates.
- Possible energy savings remain small if the mixing temperature remains above 100°C.
- Substantial energy savings can be realized if the mixing temperature remains below 100°C.
- It is however not possible in this case to eliminate all water contained in the aggregate skeleton because the water is needed to ensure the workability of the mix below 100°C by allowing it to combine momentarily with the bitumen, without any effect on mix performance. Introduction of a surfactant makes that possible.
- There is nothing to be gained by the sand being dried if the right coating is achieved otherwise.
- Some moisture content in the final mixture is not only acceptable, it actually helps. It has been observed that a water content of 0.5% at the paving site improves the mix's workability and compaction, without affecting mechanical performance.
- On the basis of fundamental thermal principles and customary conditions of bitumen use, the LEA process minimizes the amount of energy required for complete coating of the aggregate skeleton. It results in limited heating of part of the aggregate skeleton and is based on the combined action of temperature, water, and mixing energy on the components. The LEA process optimizes the physical properties of the bitumen: fluidity when hot to "wet" the coarse aggregates, "foaming" to coat the fine elements, and combining with residual water to maintain mix workability at a temperature not exceeding 80°C.
- After cooling, the mechanical performance of LEA is equivalent to that of HMA.

### LEA PRINCIPLE

As was pointed out above, the LEA process makes better use of available energy and optimizes any additional energy required. The objectives sought for the LEA process are as follows:

- Limit additional energy requirements;
- Obtain complete coating of all components, from the finest to the coarsest;

- Ensure that the final temperature of the mix is below 100°C (between 60°C and 80°C);
- Use a single binder eventually, with surfactant additives but no solvent; and
- Have a stabilized binder at the end of application.

To achieve those goals, the LEA process consists of the following:

- In the LEA process all the necessary calorific energy is brought by heating and drying only the coarse aggregates.
- LEA is produced by sequential mixing: coarse aggregates and bitumen are mixed first; wet and cold sand are introduced afterward in the mix.
- The LEA process takes advantage of the interaction between hot asphalt and water to create a final mixture at low temperature (below 100°C), with residual water content in the mix.

### LEA SEQUENTIAL MIXING PROCESS

The following describes the LEA sequential mixing process in detail (see Figure 1):

1. The aggregate skeleton is separated into two fractions: one containing the coarse aggregates and sand without fines and the other containing sand with fine elements and added filler.
2. Only the coarse aggregates and sand without fines are dried and heated; they are heated to a moderate temperature (typically 150°C). They store calorific energy that will subsequently be transferred to the cold elements of the mix.
3. The asphalt binder is heated to its recommended temperature (between 140°C and 180°C, depending on grade and nature). Specifically formulated additives are added to the asphalt binder to enable and enhance asphalt foaming and coating of wet elements. These additives make it possible, on contact between water and still-warm bitumen, to regulate expansion and prevent water stripping of bitumen already fixed on the coarse aggregates.
4. The coarse aggregates are thoroughly hot-coated with all the asphalt of the final mix. The thick and hot asphalt film clings firmly around the coarse aggregates, and it clings firmly and is well distributed around the granules and sand brought to a similar temperature.
5. The addition of wet sand at ambient temperature triggers a complex reaction that encompasses the following:
  - Asphalt binder is expanded and foaming.
  - Sand grains are encapsulated by foamed bitumen.
  - Cold sand is heated by contact.
  - Excess of water is recondensated and uniformly distributed in the mix.
  - Homogeneous mix is obtained.
  - Final mixture temperature equilibrium is reached under 100°C.

In regard to the water content of the mix, the recondensated water in the final mix, present in fine droplets, prevents mix setting, despite the low temperature. Sand moisture content can be increased, if necessary, before introduction into the mix. Additional water also can be introduced at the end of mixing to increase the workability of the mixes.

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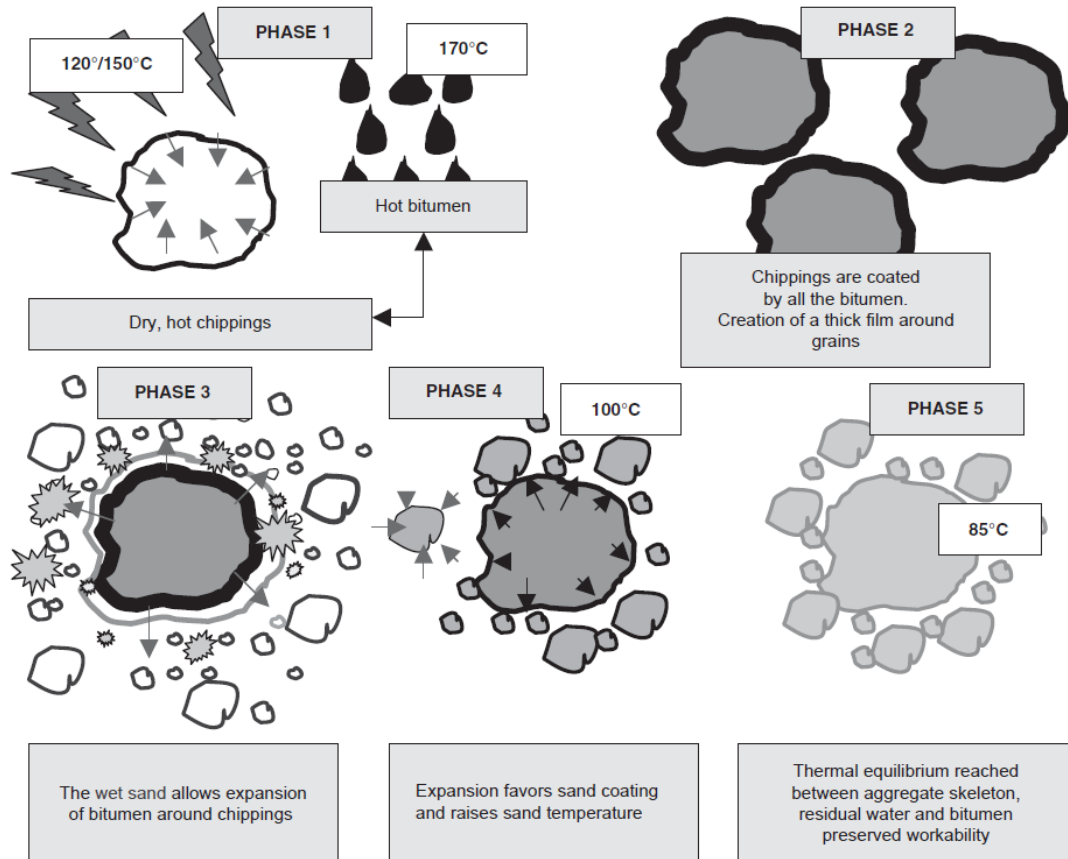


FIGURE 1 LEA technique functional diagram.

### MIX DESIGN, MANUFACTURE, AND APPLICATION

#### Laboratory Mix Design Methods for HMA Apply to Mix Design of LEA Mixes

Laboratory procedures must be adjusted to the temperature of the mixes resulting from the process. Specimen manufacturing temperatures are between 60°C and 80°C. They nevertheless allow the execution of the same types of tests used for HMA. For the LEA mix design studies, a mixing temperature of 80°C was used (see Figures 2 and 3). The following tests complied with standards (NFP 98-130 and NFP 98-138).

- Unconfined compressing test (in France, Duriez test). Cylindrical specimens are molded by vertical static pressure; there are two series of specimens, one cured at ambient temperature of 18°C during a 7-day period, the other immersed in water at 18°C. The unconfined compressive strength  $R$  (air conservation) and  $r$  (water conservation) are measured.

- Gyrotory shear test. Measure of the evolution of compaction of one specimen placed in a vertical mold and submitted to a kneading effect, according to the number of rounds of gyration.
- Wheel tracking test. Evaluation of the rutting on a slab of pre-compacted specimen. Measure of the air void and the strain under the wheel track after 30,000 passings.
- Stiffness modulus test. Performed with servo-hydraulic press. The French MAER.

Results of laboratory tests are as follows:

- The technique works with bitumen grades currently used in road building (35/50 and 50/70), but also with polymer-modified bitumen or hard binders for structuring mixes.
- Semigranular bituminous concrete-type mixes (French designation BBSG) and Class 3 asphalt road base materials (French designation GB3) manufactured according to the LEA technique make it possible to reach the same performance levels as if manufactured using the HMA technique (see Tables 1 and 2).



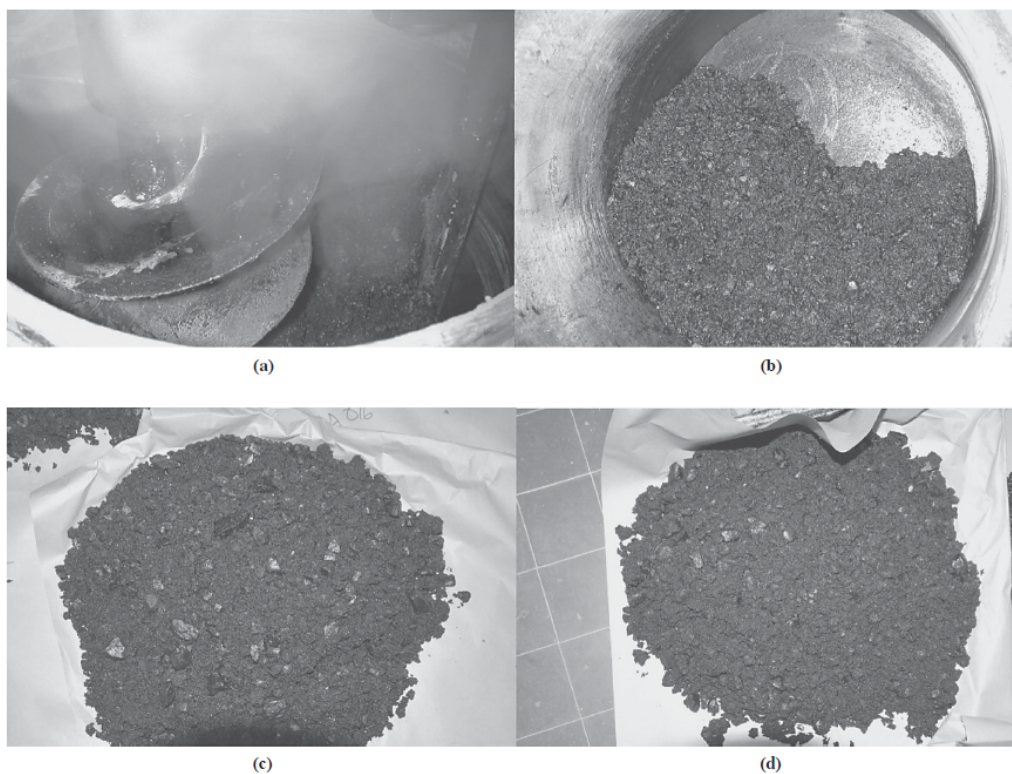


FIGURE 2 Manufacture of mix in laboratory (French TR Lab-type mixer) at 85°C: (a) and (c) mix with 30% RAP; (b) and (d) same formula with virgin aggregates.

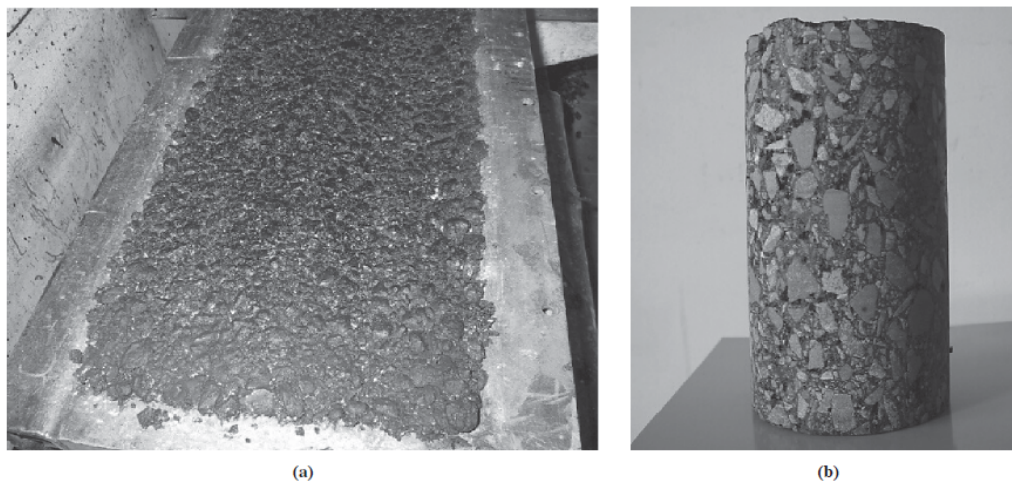


FIGURE 3 LEA mix: (a) specimen of slab intended for wheel tracking test and (b) core for stiffness modulus measure extracted from the same type of slab.



- Performance levels are reached with a manufacturing temperature of 80°C.
- Residual water content of LEAs, after cooling, is between 0.2% and 0.3%.
- Technique is applicable to the recycling of asphalt by using cold bituminous aggregate [recycled asphalt pavement (RAP)] for the partial or total replacement of the sandy cold part containing the fines.

#### First Industrial Tests—November 2003

A total of 150 tons was produced and laid down (see Figure 4). The LEA mixes were of the BBSG and GB3 type (see Table 1). They were produced by a batch plant modified for the sequential introduction of the two parts of the aggregates into the weighing system ahead of the mixer. The mixes were stored in the finished product storage hopper for more than 1 h. They were laid down by a paver-finisher and compacted by a double-roll vibrating compactor following usual application procedures. The surface appearance of the LEA mix is comparable with that of HMA, including at joint locations.

Climatic conditions corresponded to normal seasonal values, with air temperatures a few degrees above freezing in the morning. Lay-down temperature was on the order of 60°C to 70°C. Because the difference between the ambient temperature and the temperature of the LEAs is smaller than for their high-temperature counterparts, the drop in temperature with time is less significant. In addition, less soiling of equipment (transport or application) was observed, thus facilitating cleaning and greatly reducing the use of solvents. Residual water content in the mixes is lower, but near 0.5%.

Industrial tests produced the results presented below.

#### ENERGY SAVINGS AND ENVIRONMENTAL ADVANTAGES OF LEA MIXTURES—POSITION WITH RESPECT TO HMA MIXTURES

Recent life cycle analysis studies on road mixes have been carried out in the industry (French Road Contractor Association, EUROBITUME, PIARC, COLAS). They indicate the corresponding levels of energy consumption and greenhouse gas emissions for the road mixes used.

A research program on energy and environmental aspects accompanied the development of the LEA process. It was carried out at the ENSAM–Paris under the direction of Professor Y. Le Goff, who is in charge of the energy department. This study led to the development of a mathematical model simulating the energy phenomena as a function of the physical characteristics and thermal properties of the different components. The study takes into account the different mix parameters and simulates the manufacturing conditions. The model may be incorporated into the operating system of the mixing plant (Figure 5).

#### Overall Environmental Balance of LEA Process

The LEA process reduces environmental disturbances substantially and improves the safety of working personnel:

- Mixing temperature lower than 100°C (60°–90°C),
- Calorific energy required for manufacturing reduced by more than half (without employing an emulsion-type binder or one using solvents), and
- Greenhouse gas emissions (GGEs) reduced.



(a)



(b)



(c)

FIGURE 4 First industrial tests; application temperature 70°C.

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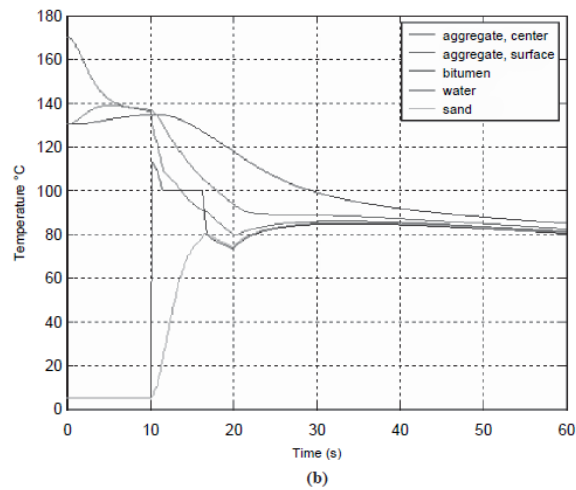
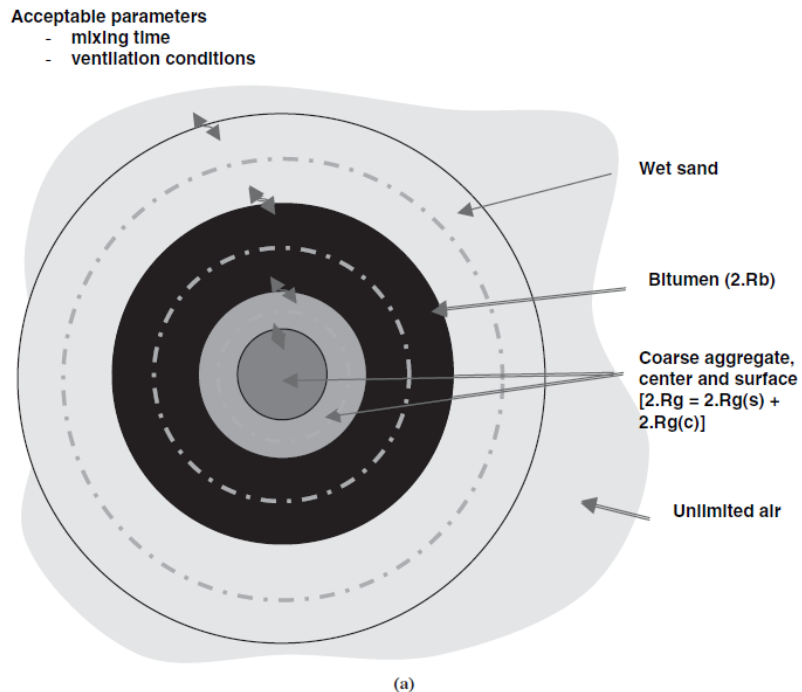


FIGURE 5 Modeling principles and premises: (a) mix modeling and (b) example of curve of transfer of temperatures between the different components of the mixture by time, according to parameters of mix.

### Comparative Heat Balance of Hot-Mix and Warm-Mix Asphalt

Only the energy required to heat the components is taken into account in what follows.

#### HMA Heat Balance

##### Assumptions

- Gradation of aggregate skeleton comprises 65% of 2/D and 35% of 0/2.
- Water content of chippings (coarse aggregates) is 1%.
- Water content of sand is 4%.
- Aggregates are heated to 150°C.
- Bitumen is 5.6 pph at 170°C.
- Ambient temperature is 10°C.

##### Calculations

For 1,000 kg of Dry Chippings

- Chipping heating energy

$$C_{\text{agg}} \times 1,000 \text{ kg} \times (150^\circ\text{C} - 10^\circ\text{C}) = 117,180.00 \text{ kJ}$$

- Water heating energy

$$C_{\text{water}} \times (1,000/990 - 1) 1,000 \times (100^\circ\text{C} - 10^\circ\text{C}) = 3,804.55 \text{ kJ}$$

- Water evaporation energy

$$L_v \times (1,000/990 - 1) 1,000 = 22,785.60 \text{ kJ}$$

- Water vapor heating energy

$$C_{\text{vap}} \times (1,000/990 - 1) 1,000 \times (150^\circ\text{C} - 100^\circ\text{C}) = 924.24 \text{ kJ}$$

- Total = 144,694.39 kJ

For 1,000 kg of Dry Sand

- Sand heating energy

$$C_{\text{agg}} \times 1,000 \text{ kg} \times (150^\circ\text{C} - 10^\circ\text{C}) = 117,180.00 \text{ kJ}$$

- Water heating energy

$$C_{\text{water}} \times (1,000/960 - 1) 1,000 \times (100^\circ\text{C} - 10^\circ\text{C}) = 15,693.75 \text{ kJ}$$

- Water evaporation energy

$$L_v \times (1,000/960 - 1) 1,000 = 94,000.00 \text{ kJ}$$

- Water vapor heating energy

$$C_{\text{vap}} \times (1,000/960 - 1) 1,000 \times (150^\circ\text{C} - 100^\circ\text{C}) = 3,812.50 \text{ kJ}$$

$$\text{Total} = 230,686.25 \text{ kJ}$$

For 1,000 kg of Dry Mix

- Coarse aggregates 65%  $\times$  144,694.39 kJ = 94,051.35 kJ
- Sand 35%  $\times$  230,686.25 kJ = 80,740.19 kJ
- Total = 174,791.54 kJ

$$\text{Total, HMA, 175 MJ}$$

#### LEA Heat Balance

##### Assumptions

- Gradation of aggregate skeleton comprises 65% of 2/D and 35% of 0/2.
- Water content of chippings is 1%.
- Water content of sand is 4%.
- Chippings heated alone to 130°C.
- Bitumen is 5.6 pph at 170°C.
- Ambient temperature is 10°C.

##### Calculations

For 1,000 kg of Dry Coarse Aggregates

- Coarse aggregates heating energy

$$C_{\text{agg}} \times 1,000 \text{ kg} \times (130^\circ\text{C} - 10^\circ\text{C}) = 100,440.00 \text{ kJ}$$

- Water heating energy

$$C_{\text{water}} \times (1,000/990 - 1) 1,000 \times (100^\circ\text{C} - 10^\circ\text{C}) = 3,804.55 \text{ kJ}$$

- Water evaporation energy

$$L_v \times (1,000/990 - 1) 1,000 = 22,785.60 \text{ kJ}$$

- Water vapor heating energy

$$C_{\text{vap}} \times (1,000/990 - 1) 1,000 \times (130^\circ\text{C} - 100^\circ\text{C}) = 554.54 \text{ kJ}$$

$$\text{Total} = 127,584.69 \text{ kJ}$$

For 1,000 kg of Dry Mix

- Coarse aggregates: 65%  $\times$  127,584.69 kJ = 82,930.05 kJ

$$\text{Total, LEA, 83 MJ}$$

#### Energy Consumption Levels Involved in Heating HMA and LEA (see Table 2)

- For HMA: 175 MJ/ton for drying and heating and
- For LEA: 83 MJ/ton for drying and heating.

The heating energy per tonne of LEA is reduced by more than 50%. The energy required for the mechanical operation of an HMA asphalt plant is of the order of 100 MJ/ton. It will be less for an LEA-equipped plant.

#### REDUCED EMISSION OF VOLATILE ORGANIC COMPOUNDS

##### Greenhouse Gas and NO<sub>x</sub> Emissions

GGEs are related to fuels consumed in heating (see Table 3). An assessment of GGEs corresponding to the heating of materials shows that GGEs, per metric ton of LEA, are reduced by more than 50%. NO<sub>x</sub> emissions also are halved (see Table 4).

##### Volatile Organic Compound Emissions

Volatile organic compound (VOC) emissions come from bitumen fumes. Among these, polycyclic aromatic hydrocarbons (PAHs) present in small quantities have an essential impact on health.

TABLE 3 Comparison of Greenhouse Gas Emissions for Tonne of HMA and Tonne of LEA

	Total Q (MJ)	Fuel Input (MJ)	GGE <sup>a</sup>			Total (kg CO <sub>2</sub> eq)
			CO <sub>2</sub> (kg CO <sub>2</sub> eq)	N <sub>2</sub> O (kg CO <sub>2</sub> eq)	CH <sub>4</sub> (kg CO <sub>2</sub> eq)	
<b>HMA mix</b>						
Dryer	172	191	15	0.148	0	15
Bitumen heat	5	6	0.44	0.004	0	0.44
Total						16
<b>LEA mix</b>						
Dryer	79	88	7	0.068	0	7
Bitumen heat	5	6	0.44	0.004	0	0.44
Total						7

<sup>a</sup>kg CO<sub>2</sub> eq = kg CO<sub>2</sub> + kg CO<sub>2</sub> eq (N<sub>2</sub>O).

PAH emissions are proportional to mix manufacturing temperature. A reduction of 10% in mix manufacturing temperature generates a reduction of these emissions by a factor of two. Reducing by more than 50% the application temperature of LEA reduces in considerable proportions the health risks involved.

#### LEA PROCESS TODAY REACHING THE INDUSTRIAL STAGE

An AMMAN batch mixing plant in operation in the Paris region, shown in Figure 6, was equipped with an LEA production-type kit at the beginning of 2005. (Schematic of a batch plant equipped for LEA processing is shown in Figure 7.)

The LEA kit includes a specific hopper that allows the metering of the amount of cold sand to be introduced into the mixer via a storage bin located over it. Also included is a device that adds water to the sand if necessary.

The coarse aggregate part of the mix design follows the usual process (metering, drying, asphalt mixing). The asphalt binder metering device is completed by a surfactant addition system. The mixing phase of the process has been modified to implement the LEA sequential mixing of hot aggregate, wet sand and bitumen, and cold filler. This installation also provides for introducing RAP directly in the mixer. The LEA mixes produced have been used for actual road paving, which is pictured in Figure 8.

#### LEA PRODUCTION EQUIPMENT SPECIFICITIES AND PERSPECTIVES

The LEA technique is applicable to current batch- and drum-mixing asphalt plants with minimum customization. Future plants specially designed for the LEA process could be optimized. In particular

the dryer and dust collection system could be downsized because only coarse aggregates are dried and heated to moderate temperature (maximum water content less than 2% and absence of fine elements). Table 5 provides information on the reduction of dust concentration in the filter.

An additional advantage of the LEA process is its ability to accommodate a higher percentage of RAP (as much as 30%) than a traditional batch plant. And the LEA process mathematical modeling makes the process ideally suitable for production automation.

#### COMPARING PROPERTIES OF LEA AND HMA MIXES

LEA mixes exhibit characteristics and performance equal to those of their hot-mix counterparts.

- LEA mixes cover the entire range of HMA mix design.
- LEA mixes use the same asphalt grades in the same proportions.
- Binder content is not a limitation, unlike with mixes based on foamed bitumen.
- LEA mixes can be stored under the same conditions as HMA mixes.
- Hauling time can be longer, typically twice as long as HMA.
- Laying and compaction of LEA mixes, at about 70°C, are done with the same equipment used for HMA mixes and under similar conditions.
- LEA mixes exhibit their final performance right after lay down.
- LEA mixes have the same appearance as HMA mixes.
- The workability of LEA mixes is compatible with manual application.
- LEA mixes are "cleaner." Less solvent or no solvent is needed to clean road-building equipment.

TABLE 4 NO<sub>x</sub> Emissions: Nitrogen Monoxide and Dioxide Constitute Main Nitrogen Oxide Emissions of Processes Using Fossil Fuel and Biomass Energy

	HMA Mix			LEA Mix		
	Total Q (MJ)	Fuel Input	NO <sub>x</sub> (g NO <sub>2</sub> eq)	Total Q (MJ)	Fuel Input	NO <sub>x</sub> (g NO <sub>2</sub> eq)
Dryer	172	191	33	79	88	15
Bitumen heat	5	6	0.96	5	6	1
Total			33			16

Q = energy consumption.



FIGURE 6 Industrial asphalt plant dedicated to LEA mixture in France.

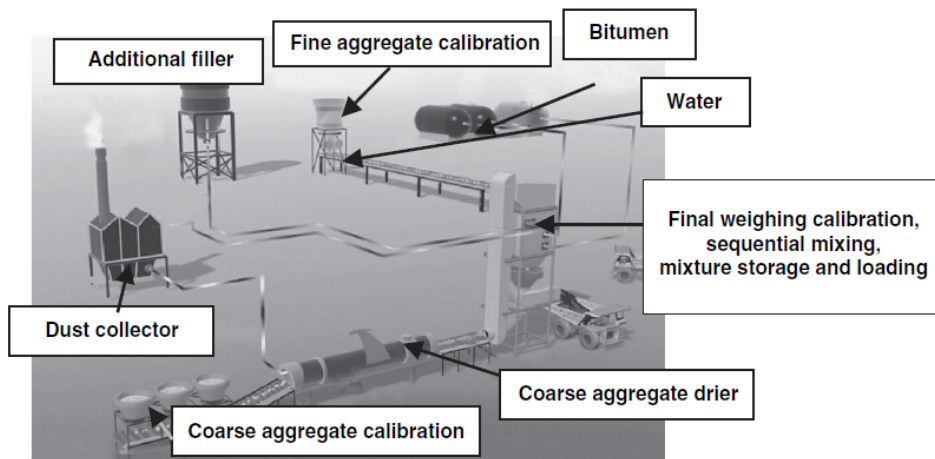


FIGURE 7 Batch plant equipped for LEA processing.



FIGURE 8 Placing LEA mixture produced by industrial plant.



TABLE 5 Comparison of Dust Emissions Corresponding to Tonne of HMA and Tonne of LEA

	Dry Matter (kg)	Fines		Dust	
		%	kg	HMA Mix	LEA Mix
Wet chippings	618.3	1	6.18	0.19	0.19
Wet sand	322.9	14	45.20	1.36	0
Total	941		51.38	1.54	0.19

## CONCLUSIONS

The studies and achievements presented show the validity and reliability of the LEA process. It is currently entering its industrial application phase. The authors are confident that time will serve merely to expand the possibilities of its application.

The LEA process represents an original response to today's challenge in energy consumption and environmental protection, without any prohibitive cost attached. LEA also improves safety and working conditions for production and paving workers and adds to the safety of motorists and pedestrians near the worksites (elimination of fog risks in rain).

Developing countries, among others, will appreciate the LEA process because it is simpler to implement and consumes less energy without sacrificing mix performance.

There is no doubt that contract awarding agencies, highway contractors, asphalt producers, and equipment manufacturers respectful of the environment will be able to further the use and development of the LEA process.

## ACKNOWLEDGMENTS

The authors acknowledge the great efficiency of the LROP laboratory crew and ENSAM-Paris thermodynamics laboratory for the performance of many tests on LEA mixes. The authors also thank Y. Le Goff

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*The Characteristics of Nonbituminous Components of Bituminous Paving Mixtures Committee sponsored publication of this paper.*

# Laboratory Investigations of Mechanical Performance of Foamed Bitumen Mixes That Use Half-Warm Aggregates

Vincent Gaudefroy, François Olard, Bogdan Cazacliu, Chantal de La Roche, Etienne Beduneau, and Jean Pierre Antoine

For several years, as environmental criteria have become stricter and stricter, new moderated-temperature mixing technologies—warm or even half-warm (less than 100°C)—have appeared. This study, realized in Laboratoire Central des Ponts et Chaussées (LCPC) in Nantes, France, within the framework of the partnership between Eiffage Travaux Publics and LCPC, aimed at determining the influence of mixing parameters (such as temperature and initial water content of the mix aggregates) on the quality of mixing and the mechanical properties of the half-warm foamed bitumen mixes compared with the traditional cold-foamed bitumen mix and hot-mix asphalt, both of which acted as reference materials. It was also a way to understand better the role of water and its influence on asphalt quality. This study highlights the influence of the increase in the aggregate temperature to elaborate mix treated with foam bitumen. This coating process allows many benefits: an increase in coating quality, compactibility, and water resistance. As a consequence, mechanical performances are improved.

For several years, as environmental criteria have become stricter and stricter, new moderated-temperature mixing technologies—warm or even half-warm (less than 100°C)—have appeared. Appia, the road branch of Eiffage Travaux Publics, has naturally been interested in this research issue and has developed in particular the patented technology of Enrobés Basse Température since the end of 2004, first in the laboratory and then quickly in almost 20 mixing plants in France and Spain in 2005–06 (1).

Another half-warm ecoprocess with foamed bitumen was studied at the Laboratoire Central des Ponts et Chaussées (LCPC) in Nantes, France, during 2005 within the framework of a scientific partnership between Eiffage Travaux Publics and LCPC Nantes. This paper presents the main results obtained within the framework of the study of these half-warm foamed bitumen mixes.

The first foamed bitumen was realized by August Jacobi, whose patent was granted in 1928 in Germany, but experts agree that Professor Ladis Csanyi, University of Iowa, was the first to intro-

duce steam in hot bitumen to mix cold aggregates (2). Real industrial tests occurred in the 1970s in Australia, but 20 more years were needed for the process to become a real implementation technique.

In France in the 1970s the company Jean Lefèvre (now Eurovia) tried to realize surface dressings with foamed bitumen but gradually changed the treated graded aggregate application area of 1971 to realize base courses (2, 3). At that time, aggregates of the mix were at ambient temperature, not dried or warmed up. In 1989, the process was extended to the treatment of gravel, and the first studies in the laboratory were performed (4).

Today, several authors have divided opinions about the influence of the bitumen composition and origin on its ability to foam (5, 6).

The study presented in this paper, carried out in LCPC Nantes, aimed at determining the influence of the mixing parameters (such as temperature and initial water content of the mix aggregates) on the quality of mixing and the mechanical properties of the half-warm foamed bitumen mixes compared with the traditional cold-foamed bitumen mix and hot-mix asphalt (HMA), both of which acted as reference materials. It was also a way to better understand the role of water and its influence on asphalt quality.

## EXPERIMENTAL MATERIALS AND TECHNIQUES

### Materials

The chosen grading formula is a dense bituminous mixture, 0/10 (French designation BBSG), with diorite aggregates coming from the French Noubleau quarry. In relation to the binder, two bitumens from different origins and having the same grade (35/50), a naphthenic one, 35/50 (penetration: 47; ring and ball (R&B) softening point: 52.2°C), and a paraffinic one, 35/50 (penetration: 45; R&B: 53°C), have been studied. It was a way to determine whether the bitumen origin and its foaming abilities could influence the mixing quality and whether, in contrast, the chosen foam-mixing process could fit different bitumens.

### Mixing Principle

During the study, wet aggregates, either cold or warmed (temperature under 100°C) were mixed with a foamed bitumen. The foamed bitumen was composed of bitumen (95% in mass) and of water. The foaming device used was the Foamlab developed by the SAE

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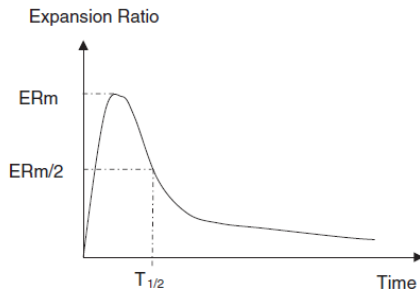


FIGURE 1 Schematic evolution expansion coefficient of a foamed bitumen with time.

Company. In the mixing process (60 s at 110 rpm), the two liquids (water and bitumen) are closely mixed under pressure and then injected through an injector to the outside. Water vaporizes instantaneously, creating bitumen films and leading to foam formation. The foam is directly sprayed into a mixer that contains the aggregates at controlled temperature and humidity. Foam is characterized by a maximum expansion ratio ( $ER_m$ ) and a half-life period ( $T_{1/2}$ ), as illustrated in Figure 1.

When the foam is injected upon the wet aggregates, the bitumen mixes with thin particles to form mastic particles (Figure 2). These particles remain dissociated from the aggregates as long as a film of water avoids coalescence (7).

#### Experimental Techniques

The used mixer was an SAE (Fayat Group) prototype (Figure 3). It included a discontinuous mixer with horizontal shafts. This type of mixer is common for unbound materials and well adapted to the bitumen foaming system. The mixer included double horizontal shafts, seven handles per shaft and two blades per handle. The blades had a surface of  $240 \times 150$  mm and were inclined at  $38^\circ$  to favor the material flow along the shaft axes. The first ones and the last ones

are retroinclined. However, the mixer was not thermally regulated, a situation that led to high energy losses.

Aggregate temperature ( $20^\circ\text{C}$ ,  $70^\circ\text{C}$ , or  $90^\circ\text{C}$ ), water content (1% to 7%), and mix bitumen content [3.5% extractables (ext) or 5.6% ext (or pph)] are the studied parameters.

Taking into account the low number of bibliographic references dealing with foamed bitumen asphalts, the researchers performed preliminary tests at different aggregate water contents—1%, 3%, 5%, and 7%—for temperatures of  $20^\circ\text{C}$ ,  $70^\circ\text{C}$ , or  $90^\circ\text{C}$ . These materials were characterized during compaction, but the coating appearance (Figures 4 and 5) was preferably used to determine the optimal water content (5% moisture in aggregates). Then, additional mixings were performed with this water content, on which a more elaborated study of mechanical characteristics was carried out.

#### Gyratory Shear Compactor Tests

To characterize the mechanical performance of this material, different tests were conducted. For all the tested modalities, a gyratory shear compactor (GSC) test according to EN 12697-31 standard (three specimens; 200 rotations, angle of  $1^\circ$ , and heating device set at mixing temperature) was performed. During the GSC tests, a suction system placed under the heating device allowed the recovery of water in a measuring test tube. The amount of water drained from the mix during the compaction in relation to the number of gyrations was also measured.

#### Direct Compressive Strength Test and Stripping Resistance

Optimized formulas were subjected to the altered (7) Duriez test (NFP 98 251-4), in which 12 cylindrical specimens were compacted. In this test, a first set of specimens is stored for 8 days in water at a temperature of  $18^\circ\text{C}$ , and the second set is stored in an oven at a temperature of  $18^\circ\text{C}$  and 50% moisture. Then, the compressive strength of the two specimens sets ( $r$ : compressive strength of specimens stored in water;  $R$ : compressive strength of specimens stored in air) are measured, and the  $r/R$  ratio characterizes stripping resistance of the material.

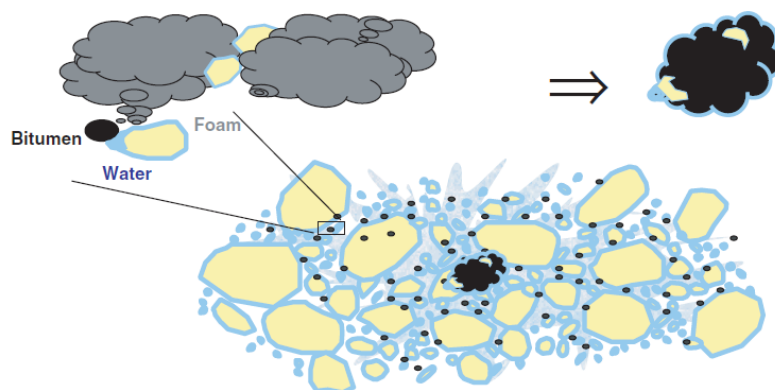


FIGURE 2 Principle of foam formation and wet aggregates and foam mixing.

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FIGURE 3 Views of (a) SAE mixer and (b) injection of foam into mixer.

#### *Characterization of Mix Composition by Means of Rouen Method*

The Rouen test (NF EN 12697-1) makes the control of the asphalt mixture formula and its binder content possible. A known asphalt mass is mixed and shaken with a known tetrachloroethylene quantity until the bitumen is totally dissolved. Then, a little solvent is put in a centrifuge to separate the fines. Finally, with the solvent evaporation, the asphalt mix's binder content is deduced. Aggregates are cleaned with solvent and sieved in a column, which allows plotting of the asphalt grading curve and verification of its conformity with the theoretical grading curve.

#### *Complex Modulus*

Hydrocarbon materials are viscoelastic and thermodependent. Therefore, their behavior varies according to the loading frequency and the



FIGURE 4 Specimens during mixing.

temperature. Contrary to the elastic materials characterized by their Young's modulus, bituminous materials need an  $E^*$  complex number characteristic for fixed temperature and frequency [15°C and 10 Hz, according to the French pavement design method (8)]. Complex modulus measurements have been performed on parallelepipedic samples of 50 × 50 × 125 mm according to the European standard NF EN 12697-26 but with bigger sizes so as to avoid material losses. Specimens presented on Figure 6 are sawed after 1 week and then tested 2 weeks later.

## LABORATORY TESTS RESULTS

### Coating Quality

Visually, it is apparent that coating of bituminous loose mix is not complete and depends on the process parameters, especially water content. As an example, uncoated aggregates and large bituminous mastic particles that represent a mix of filler and bitumen were observed for a water content of 3.5%. However, for a higher water content (more than 4.5%), fine and well-dispersed bituminous mastic particles were noticed in the mix.

This phenomenon could be explained by the role of water in gravel treated with foamed bitumen. Because of the hydrophobic nature of bitumen, water enhances the workability of bituminous mixes and acts as lubricant in the mix. It prevents the bitumen from sticking to the aggregates, and then connections between the bitumen and the aggregates are low. However, the finest portion of the aggregate, due to its great specific surface, absorbs a significant part of the binder and leads to the formation of mastic particles. The low connection numbers between the bitumen and the largest aggregates indicate that the cohesion evolution is slow for the mix stored in hermetic conditions. In the same way, bituminous mastic particles are not connected to each other during the mixing process; this condition leads to a homogeneous distribution of the mastic in the material.

Finally, the fact that the aggregate temperature is increasing limits the viscosity drop of the bitumen (by an important loss of temperature) when foamed bitumen and aggregates are mixed together. The



FIGURE 5 Appearance of product for two initial aggregates moistures: (a) mixing at 70°C and 0.84% water and (b) mixing at 70°C and 7% water.

consequence is a coating of the largest aggregates by the remaining bitumen during the mixing process.

#### Workability

The manufactured material shows good workability with aggregate temperature between 70°C and 90°C (Figure 7). In accordance with standard symbols, the dotted lines in Figure 3 define the HMA (BBSG 0/10) void content value ranges to be reached at 10 and 60 rotation numbers. The best results obtained lead to a void content of 7% at 200 gyrations for a binder content of 5.6% ext. Samples manufactured at 20°C are too stiff to be compacted by means of classic techniques used in industry. In fact, at room temperature, bitumen viscosity is extremely low and therefore requires high compaction energy. In the same way, for a binder content of 3.5% ext,

material workability decreases: the void content is about 10% to 12% at 200 gyrations due to the low thickness of the bitumen film, which limits aggregate reorganization in the mix.

In contrast, it seems difficult to link the final void content and the amount of water drained from the mix during compaction for similar initial water content. This could perhaps be explained by a water-aggregate interaction. However, this hypothesis, which could not be confirmed during this study, is still to investigate. It could also be possible that some results were distorted by the blinding of the water collection system because of the presence of filler in the mix.

#### Direct Compressive Strength and Stripping Resistance

The best results were obtained for the same process parameters in the case of the Duriez test. In these conditions, average values for  $R$  and  $r/R$  are respectively equal to 9 MPa and 0.5 (Figure 8).

An increase in the aggregate temperature enhances compactibility. Compacted materials then present a closed texture surface that increases resistance to stripping. Meanwhile, performance of the mix treated with foam bitumen are low compared with that of hot bituminous mixes. For a BBSG 0/10, the  $r/R$  value is higher than 0.75. For a gravel emulsion cold mix, the  $r/R$  value must be higher than 0.55. According to HMA standards,  $r/R$  values are poor with respect to control mix. In contrast, however, the Duriez test results of the mix treated with foam bitumen are equivalent to the ones of cold mixes. A 5% moisture content allows the mix to be optimized to get a compromise that enhances coating quality, workability, and mechanical performances.

To complete previous experiments, two materials were manufactured with aggregates respectively at 20°C and 90°C for a bitumen content of 4.3% ext (naphthenic origin). The Duriez test results are reported in Table 1.

#### Complex Modulus Results

Figure 9 shows the evolution of complex modulus with frequency for various aggregate temperatures. These tests have been performed at 15°C on an optimized formula [binder content at 5.6% ext from naphthenic bitumen (Table 2)].



FIGURE 6 Complex modulus specimen.

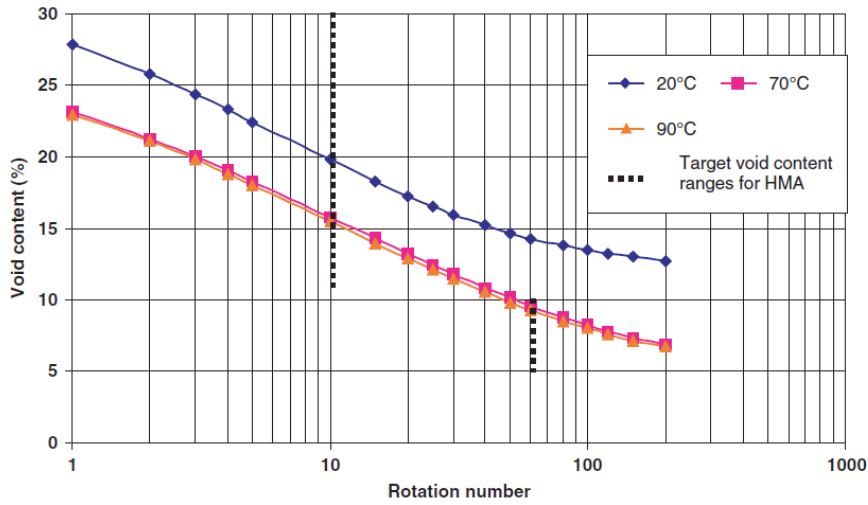


FIGURE 7 Workability at 20°C, 70°C, and 90°C (aggregate temperatures) for the naphthenic bitumen 35/50 (initial water content of aggregates: 5%; binder content: 5.6% ext).

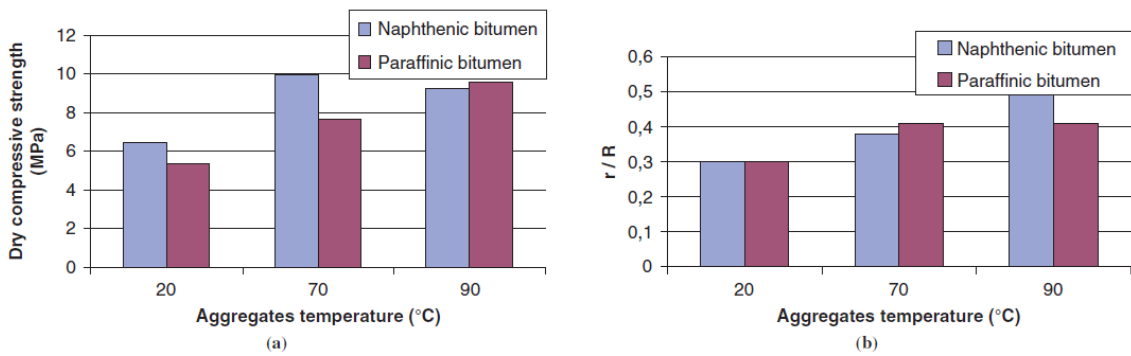


FIGURE 8 Evolution of dry compressive strength and  $r/R$  ratio with aggregate temperature for the two studied bitumens: (a) dry compressive strength evolution and (b)  $r/R$  ratio evolution.

TABLE 1 Results of Duriez Tests on Formula with Aggregate Water Content of 5%

Aggregates Temperature (°C)	Bitumen Origin	Binder Content (% ext)	$R$ (MPa)	$r$ (MPa)	$r/R$	Bulk Density (g/cm <sup>3</sup> )
20	Naphthenic	4.35	5.75	1.67	0.29	2.395
90	Naphthenic	4.31	9.35	3.44	0.37	2.383
20	Naphthenic	5.76	6.45	1.95	0.30	2.398
70	Naphthenic	5.36	9.97	3.75	0.38	2.482
90	Naphthenic	Data not available	9.24	4.62	0.50	2.488
20	Paraffinic	5.67	5.36	1.63	0.30	2.378
70	Paraffinic	5.54	7.66	3.08	0.41	2.466
90	Paraffinic	5.41	9.58	3.92	0.41	2.469

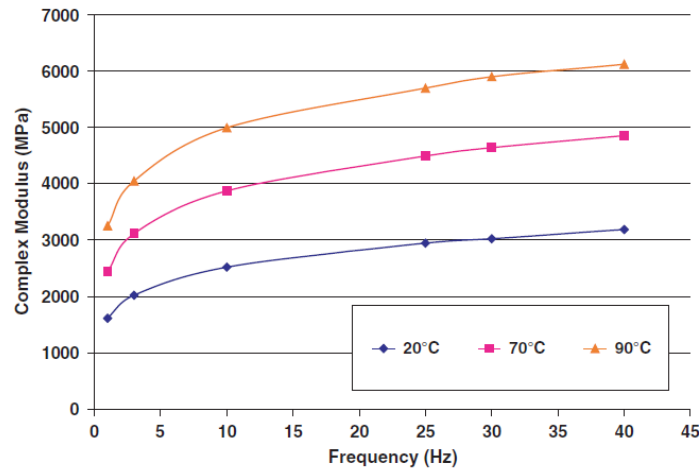


FIGURE 9 Variation of complex modulus with frequency at 15°C, for different mixing temperatures of the specimens (binder content: 5.6% ext).

Table 2 shows that an increase in mixing temperature favors a higher complex modulus. As noted earlier, workability is enhanced by temperature, a situation that leads to a cohesion increase. Furthermore, residual water content is lower and is accompanied by an increase in the mechanical properties of the mix.

One can conclude that aggregate temperature and water content effects highlighted throughout the complex modulus characterization are in agreement with gyratory shear compactor (GSC) and Duriez results (Table 1). However, complex modulus values are lower for samples treated with foamed bitumen than for hot asphalt mixes. (Complex modulus for BBSG 0/10, Class 1, must be higher than 5,400 MPa at 15°C and 10 Hz according to NF EN 13108-1.)

A temperature increase from 20°C to 90°C does not lead to more aging of the bitumen, as the warm-mix process already implies that the bitumen is heated to 160°C.

## CONCLUSIONS AND FUTURE PROSPECTS

This study highlights the advantage of increasing the aggregate temperature so as to elaborate a higher-performance mix treated with foam bitumen: improved coating, higher workability, and higher

stripping resistance. As a consequence, mechanical performances are improved.

First, researchers considered preparing cold specimens at room temperature and then warming them up before compaction at 90°C. However, this experiment has not yet been realized.

Second, the aggregate temperatures investigated were lower than 100°C and did not significantly modify the initial moisture content of the aggregates. The initial moisture of the aggregates strongly influenced coating quality. In this process, the water acted to disperse the bitumen in the mix and led to a homogeneous material. In contrast, water limited the bitumen cohesion on aggregates. In fact, high residual water content led to mechanical performance that was lower than that for hot bituminous mixes (in particular, the Duriez and complex-modulus results). For users to benefit from energy savings of half-warm materials, process improvements are needed to achieve hot-mix performance. However, actual measured performances are sufficient for foamed-bitumen mixes manufacturing.

Finally, mechanical properties obtained from paraffinic and naphthenic bitumens are similar (except for stripping resistance, which is higher with naphthenic bitumen). However, the ability to foam seems to depend on bitumen origin and composition. This phenomenon confirms results reported in the literature (6). It indicates that

TABLE 2 Main Characteristics of Half-Warm Mixes and Reference Hot Mix with 50/70 Naphthenic Bitumen

	Reference Hot Mix	Half-Warm Mix with Aggregates Temperature of 20°C	Half-Warm Mix with Aggregates Temperature of 70°C	Half-Warm Mix with Aggregates Temperature of 90°C
Mix temperature after mixing (°C)	160	28	47	70
Residual aggregate moisture (%)	0	4.48	4.31	4.17
GSC void content after 60 gyrations (%)	9.3	14	10.9	9
Duriez <i>R</i> (MPa)	11.7	5.36	7.66	9.58
Duriez <i>r</i> (MPa)	1.6	1.63	3.08	3.82
Duriez <i>r/R</i>	0.98	0.3	0.41	0.41
Complex modulus at 15°C-10Hz (MPa)	>5,500 (standard)	2,520	3,875	5,000

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the actual foamability characterization test partially fails to determine with accuracy the coating capacity of bitumen.

On the basis of these results, the process studied allows improvement in coating, with performance between that of HMA and that of cold-mix asphalt. In the case of in-place recycling, processes with cold aggregates treated with foamed bitumen and with emulsified bitumen are competitive. In addition, in a production plant, it is possible to warm up aggregates and then achieve better performance. In this case, aggregates coated with foamed bitumen mixes are manufactured at lower temperature than HMAs, a situation that provides an economic benefit by reducing the amount of heating required to produce the mixes.

Other tracks may be investigated to reduce mixing temperature and target HMA performance. This is the case with warm-mix asphalts, such as low-energy asphalt, at approximately 95°C, which does not use foamed bitumen but only bitumen (with additives) emulsified during mixing with warmed and partially dried aggregates (9–11).

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(54) **Procédé pour fabriquer un enrobé bitumineux**

(57) Le procédé comprend une étape de séchage des granulats suivie d'une étape d'enrobage de la totalité de ceux-ci par un liant bitumineux chaud.  
 Selon l'invention, l'étape de séchage est réalisée

dans des conditions laissant subsister une fraction de l'humidité initiale des granulats.  
 Application à la construction routière.

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## Description

- 5 [0001] L'invention concerne un procédé pour fabriquer un enrobé comprenant des fragments solides enrobés d'un liant, le procédé comprenant une étape de séchage des fragments solides suivie d'une étape d'enrobage de la totalité des fragments solides par du liant, en particulier par du liant chaud.
- [0002] Par "fragments solides", on entend ici tous fragments solides utilisables pour la réalisation d'enrobés notamment pour la construction routière, comprenant notamment les granulats minéraux naturels, et les agrégats d'enrobés résultant du recyclage des matériaux récupérés lors de la réfection des routes.
- 10 [0003] On entend par "liant" tout liant hydrocarboné d'origine fossile ou végétale utilisable pour la réalisation d'enrobés, notamment du bitume pur ou additionné de fluxants et/ou de fluidifiants et/ou modifié par adjonction de polymères, ce liant pouvant éventuellement se présenter sous la forme d'une émulsion ou d'une mousse.
- [0004] Dans le procédé classique d'enrobage à chaud, l'étape de séchage est réalisée de manière à éliminer sensiblement en totalité l'humidité des granulats, ce qui nécessite une dépense d'énergie considérable en raison de la grande quantité d'eau contenue initialement dans les granulats. De plus, le séchage et l'élévation de température entraînent un dégagement de fumées contenant de la vapeur d'eau chargée en poussières. Compte tenu de leur grande quantité, ces fumées sont difficiles à traiter et la mise en oeuvre du procédé entraîne donc une pollution non négligeable de l'environnement.
- 15 [0005] Pour résoudre ce problème, EP 1 469 038 A propose de faire porter l'étape de séchage sur une première partie des granulats, sensiblement dépourvue de fines, d'enrober de bitume chaud cette première partie, et d'ajouter ensuite au mélange ainsi obtenu une deuxième partie des granulats, comprenant des sables et des fines.
- [0006] Ce procédé nécessite de disposer de deux fractions de granulats, l'une dépourvue de fines et l'autre contenant des fines, ce qui peut impliquer des difficultés. De plus, ce procédé ne peut être mis en oeuvre efficacement dans certaines installations d'enrobage existantes, et peut donc nécessiter un aménagement coûteux de celles-ci.
- 20 [0007] Le but de l'invention est d'éliminer tout ou partie de ces inconvénients.
- [0008] L'invention vise notamment un procédé du genre défini en introduction, et prévoit que l'étape de séchage est réalisée dans des conditions laissant subsister une fraction de l'humidité initiale des fragments solides.
- 25 [0009] Des caractéristiques optionnelles de l'invention, complémentaires ou de substitution, sont énoncées ci-après:
- L'étape de séchage porte seulement sur une première partie des fragments solides, qui est ensuite mélangée, avant l'étape d'enrobage, à la partie restante ayant conservé son humidité initiale.
  - L'étape de séchage est réalisée à une température supérieure à 100 °C, de préférence à une température d'environ 130 °C.
  - 35 - La limite supérieure de la granulométrie de ladite partie restante est inférieure, égale ou supérieure à la limite supérieure de la granulométrie de ladite première partie.
  - La limite supérieure de la granulométrie de ladite partie restante est inférieure ou égale à la limite inférieure de la granulométrie de ladite première partie.
  - 40 - La granulométrie de ladite partie restante est choisie parmi 0/2 et 0/4.
  - L'étape de séchage porte sur la totalité des fragments solides et est réalisée dans des conditions laissant subsister une fraction de l'humidité de ceux-ci.
  - 45 - L'étape de séchage est réalisée à une température inférieure à 100 °C, de préférence à une température d'environ 90 °C.
  - Une quantité contrôlée d'eau est ajoutée aux fragments solides entre les étapes de séchage et d'enrobage et/ou après l'étape d'enrobage.
  - 50 - Le liant est mis en oeuvre à une température comprise entre 100 et 200 °C, de préférence d'environ 160 °C, ou entre 30 et 90 °C dans le cas d'un liant sous forme d'émulsion.
- 55 [0010] Dans une variante du procédé selon l'invention, l'étape de séchage porte seulement sur une première partie des fragments solides, qui est ensuite enrobée, puis la partie restante, ayant conservé son humidité initiale, est ajoutée au mélange obtenu, au moins la première partie contenant des fines.
- [0011] L'invention est illustrée ci-après par des exemples. Dans ces exemples, les fragments solides sont des granulats

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minéraux de granulométrie 0/10 mm, le liant est un bitume de pénétrabilité 35/50 selon la norme NF EN 1426 et le séchage est effectué à 130 °C lorsqu'il porte seulement sur une première partie des granulats et à 90 °C lorsqu'il porte sur la totalité des granulats. Les proportions indiquées sont en masse.

5 Exemple 1

**[0012]** Les fragments solides sont constitués de 98 % de granulats de granulométrie 0/10 mm et de 2 % de fines calcaires. Les 98 % de granulats 0/10 sont constitués de porphyre et présentent la répartition granulométrique suivante :

10	6/10	44
	2/6	22
	0/2	32

15 **[0013]** Les fractions 6/10 et 2/6 et les fines sont mélangées et séchées à 130 °C, puis on ajoute à température ambiante la fraction 0/2 dont le taux d'humidité est de 4 %. On introduit ensuite 5,6 % de bitume 35/50 à 160 °C pour réaliser l'enrobage.

20 Exemple 2

**[0014]** La composition des fragments solides et la quantité de liant sont les mêmes que dans l'exemple 1.

**[0015]** Le séchage est effectué sur la totalité des fragments solides, après quoi on ajoute 1,5 % d'eau par rapport à la masse de ceux-ci, avant de réaliser l'enrobage par introduction du liant à 160 °C.

25 Exemple 3

**[0016]** La nature et la répartition granulométrique des fragments solides et la quantité de liant sont les mêmes que dans les exemples 1 et 2.

30 **[0017]** L'ensemble des fragments solides est divisé de manière homogène en une première partie représentant 65 % en masse et une seconde partie représentant 35 % en masse. La première partie est séchée à 130 °C, puis on ajoute à température ambiante la seconde partie dont le taux d'humidité est de 4 %. On introduit ensuite 5,6 % de bitume 35/50 à 160 °C pour réaliser l'enrobage.

35 Exemple 4

**[0018]** La nature et la répartition granulométrique des fragments solides et la quantité de liant sont les mêmes que dans les exemples 1, 2 et 3.

40 **[0019]** L'ensemble des fragments solides est divisé de manière homogène en une première partie représentant 80 % en masse et une seconde partie représentant 20 % en masse. Le séchage est effectué sur la première partie, qui est ensuite enrobée par le liant introduit à 160 °C, après quoi on ajoute la seconde partie qui est à température ambiante et qui présente un taux d'humidité de 4 %.

45 Exemple 5

**[0020]** Dans cet exemple, les fragments solides sont composés en masse de 2 % de fines calcaires, comme dans les exemples précédents, et de 98 % de granulats silico-calcaires de granulométrie 0/10 mm, la répartition granulométrique de ces derniers étant la suivante :

50	6/10	29
	4/6	28
	0/4	41

55 **[0021]** On procède comme dans l'exemple 3, à ceci près que les deux parties des fragments solides n'ont pas la même composition granulométrique, la première partie comprenant la totalité des fractions 6/10 et 4/6 et des fines et une quote-part de la fraction 0/4 correspondant à 16 % du total des fragments solides, et la seconde partie étant constituée par les 25 % restant de la fraction 0/4, présentant un taux d'humidité de 4 %. Par ailleurs, la quantité de liant est portée à 5,9 % de la masse des fragments solides.

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[0022] Le tableau ci-après donne pour chacun des exemples la température finale de l'enrobé et diverses caractéristiques de celui-ci. À titre de comparaison, les données correspondantes sont fournies pour des enrobés de référence obtenus à partir des mêmes composants par le procédé classique consistant à sécher la totalité des fragments solides à 160 °C avant l'enrobage.

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Exemple	1	2	3	4	Référence
T finale (°C)	≈ 85	≈ 90	≈ 85	≈ 90	160
Température PCG	95	95	95	95	160
PCG %	7,1	6,8	7,1	8,8	9,3
% d'eau	0,3	0,3	0,4	0,2	-
% de vides	5,7	7,4	9,2	7,4	6,2
Orniérage	6,4	9,6	6,2	5,2	6,4
r/R	0,84	0,81	0,80	0,82	0,90

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Exemple	5	Référence
T finale (°C)	≈ 90	160
Température PCG	95	160
PCG %	9,9	10,8
% d'eau	0,1	-
% de vides	8,0	5,8
Orniérage	5,7	6,4
r/R	0,70	0,88

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[0023] Dans le tableau :

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- "Température PCG" représente la température en °C à laquelle l'enrobé est porté pour réaliser l'essai PCG selon la norme française P 98-252,
- "PCG %" représente le pourcentage de vides obtenu pour 60 girations lors de l'essai à la presse à cisaillement giratoire (PCG) selon la norme française P 98-252,
- "% d'eau" représente la teneur en eau en masse de l'enrobé à l'issue de l'enrobage,
- "% de vides" représente le pourcentage de vides initial de l'éprouvette soumise à l'essai d'orniérage ,
- "Orniérage" représente le pourcentage de vides après 30 000 cycles lors de l'essai d'orniérage selon la norme française P 98-253-1,
- "r/R" représente le rapport r/R obtenu lors de l'essai Duriez selon la norme française P 98-251-1.

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#### Revendications

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1. Procédé pour fabriquer un enrobé comprenant des fragments solides enrobés d'un liant, le procédé comprenant une étape de séchage des fragments solides suivie d'une étape d'enrobage par adjonction exclusivement d'un liant à la totalité des fragments solides, **caractérisé en ce que** l'étape de séchage est réalisée dans des conditions laissant subsister une fraction de l'humidité initiale des fragments solides.
2. Procédé selon la revendication 1, dans lequel l'étape de séchage porte seulement sur une première partie des fragments solides, qui est ensuite mélangée, avant l'étape d'enrobage, à la partie restante ayant conservé son humidité initiale.
3. Procédé selon la revendication 2, dans lequel l'étape de séchage est réalisée à une température supérieure à 100

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°C, de préférence à une température d'environ 130 °C.

4. Procédé selon la revendication 1, dans lequel l'étape de séchage porte sur la totalité des fragments solides et est réalisée dans des conditions laissant subsister une fraction de l'humidité de ceux-ci.

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5. Procédé selon la revendication 4, dans lequel l'étape de séchage est réalisée à une température inférieure à 100 °C, de préférence à une température d'environ 90 °C.

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6. Procédé selon l'une des revendications précédentes, dans lequel une quantité contrôlée d'eau est ajoutée aux fragments solides entre les étapes de séchage et d'enrobage et/ou après l'étape d'enrobage.

7. Procédé selon l'une des revendications précédentes, dans lequel le liant est d'origine fossile ou végétale

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8. Procédé selon l'une des revendications précédentes, dans lequel le liant est mis en oeuvre à une température comprise entre 100 et 200 °C, de préférence d'environ 160 °C, ou entre 30 et 90 °C dans le cas d'un liant sous forme d'émulsion.

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(54) **Procédé pour fabriquer un enrobé bitumineux avec pré-enrobage**

(57) On sèche et on enrobe une première partie des fragments solides, contenant des fines, après quoi la partie restante des fragments solides, ayant conservé son

humidité initiale, est ajoutée au mélange obtenu.  
Application à la construction routière.

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## Description

5 **[0001]** L'invention concerne un procédé pour fabriquer un enrobé comprenant des fragments solides enrobés d'un liant, le procédé comprenant une étape de séchage des fragments solides suivie d'une étape d'enrobage de la totalité des fragments solides par du liant, en particulier par du liant chaud.

**[0002]** Par "fragments solides", on entend ici tous fragments solides utilisables pour la réalisation d'enrobés notamment pour la construction routière, comprenant notamment les granulats minéraux naturels, et les agrégats d'enrobés résultant du recyclage des matériaux récupérés lors de la réfection des routes.

10 **[0003]** On entend par "liant" tout liant hydrocarboné d'origine fossile ou végétale utilisable pour la réalisation d'enrobés, notamment du bitume pur ou additionné de fluxants et/ou de fluidifiants et/ou modifié par adjonction de polymères, ce liant pouvant éventuellement se présenter sous la forme d'une émulsion ou d'une mousse.

**[0004]** Dans le procédé classique d'enrobage à chaud, l'étape de séchage est réalisée de manière à éliminer sensiblement en totalité l'humidité des granulats, ce qui nécessite une dépense d'énergie considérable en raison de la grande quantité d'eau contenue initialement dans les granulats. De plus, le séchage et l'élévation de température entraînent un dégagement de fumées contenant de la vapeur d'eau chargée en poussières. Compte tenu de leur grande quantité, ces fumées sont difficiles à traiter et la mise en oeuvre du procédé entraîne donc une pollution non négligeable de l'environnement.

15 **[0005]** Pour résoudre ce problème, EP 1 469 038 A propose de faire porter l'étape de séchage sur une première partie des granulats, sensiblement dépourvue de fines, d'enrober de bitume chaud cette première partie, et d'ajouter ensuite au mélange ainsi obtenu une deuxième partie des granulats, comprenant des sables et des fines.

**[0006]** Ce procédé nécessite de disposer de deux fractions de granulats, l'une dépourvue de fines et l'autre contenant des fines, ce qui peut impliquer des difficultés. De plus, ce procédé ne peut être mis en oeuvre efficacement dans certaines installations d'enrobage existantes, et peut donc nécessiter un aménagement coûteux de celles-ci.

**[0007]** Le but de l'invention est d'éliminer tout ou partie de ces inconvénients.

25 **[0008]** L'invention vise notamment un procédé du genre défini en introduction, et prévoit que l'étape de séchage est réalisée dans des conditions laissant subsister une fraction de l'humidité initiale des fragments solides.

**[0009]** Des caractéristiques optionnelles de l'invention, complémentaires ou de substitution, sont énoncées ci-après:

30 - L'étape de séchage porte seulement sur une première partie des fragments solides, qui est ensuite mélangée, avant l'étape d'enrobage, à la partie restante ayant conservé son humidité initiale.

- L'étape de séchage est réalisée à une température supérieure à 100 °C, de préférence à une température d'environ 130 °C.

35 - La limite supérieure de la granulométrie de ladite partie restante est inférieure, égale ou supérieure à la limite supérieure de la granulométrie de ladite première partie.

- La limite supérieure de la granulométrie de ladite partie restante est inférieure ou égale à la limite inférieure de la granulométrie de ladite première partie.

40 - La granulométrie de ladite partie restante est choisie parmi 0/2 et 0/4.

- L'étape de séchage porte sur la totalité des fragments solides et est réalisée dans des conditions laissant subsister une fraction de l'humidité de ceux-ci.

45 - L'étape de séchage est réalisée à une température inférieure à 100 °C, de préférence à une température d'environ 90 °C.

50 - Une quantité contrôlée d'eau est ajoutée aux fragments solides entre les étapes de séchage et d'enrobage et/ou après l'étape d'enrobage.

- Le liant est mis en oeuvre à une température comprise entre 100 et 200 °C, de préférence d'environ 160 °C, ou entre 30 et 90 °C dans le cas d'un liant sous forme d'émulsion.

55 **[0010]** Dans une variante du procédé selon l'invention, l'étape de séchage porte seulement sur une première partie des fragments solides, qui est ensuite enrobée, puis la partie restante, ayant conservé son humidité initiale, est ajoutée au mélange obtenu, au moins la première partie contenant des fines.

**[0011]** L'invention est illustrée ci-après par des exemples. Dans ces exemples, les fragments solides sont des granulats

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minéraux de granulométrie 0/10 mm, le liant est un bitume de pénétrabilité 35/50 selon la norme NF EN 1426 et le séchage est effectué à 130 °C lorsqu'il porte seulement sur une première partie des granulats et à 90 °C lorsqu'il porte sur la totalité des granulats. Les proportions indiquées sont en masse.

5 Exemple 1

**[0012]** Les fragments solides sont constitués de 98 % de granulats de granulométrie 0/10 mm et de 2 % de fines calcaires. Les 98 % de granulats 0/10 sont constitués de porphyre et présentent la répartition granulométrique suivante :

10	6/10	44
	2/6	22
	0/2	32

15 **[0013]** Les fractions 6/10 et 2/6 et les fines sont mélangées et séchées à 130 °C, puis on ajoute à température ambiante la fraction 0/2 dont le taux d'humidité est de 4 %. On introduit ensuite 5,6 % de bitume 35/50 à 160 °C pour réaliser l'enrobage.

Exemple 2

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**[0014]** La composition des fragments solides et la quantité de liant sont les mêmes que dans l'exemple 1.

**[0015]** Le séchage est effectué sur la totalité des fragments solides, après quoi on ajoute 1,5 % d'eau par rapport à la masse de ceux-ci, avant de réaliser l'enrobage par introduction du liant à 160 °C.

25 Exemple 3

**[0016]** La nature et la répartition granulométrique des fragments solides et la quantité de liant sont les mêmes que dans les exemples 1 et 2.

30 **[0017]** L'ensemble des fragments solides est divisé de manière homogène en une première partie représentant 65 % en masse et une seconde partie représentant 35 % en masse. La première partie est séchée à 130 °C, puis on ajoute à température ambiante la seconde partie dont le taux d'humidité est de 4 %. On introduit ensuite 5,6 % de bitume 35/50 à 160 °C pour réaliser l'enrobage.

Exemple 4

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**[0018]** La nature et la répartition granulométrique des fragments solides et la quantité de liant sont les mêmes que dans les exemples 1, 2 et 3.

40 **[0019]** L'ensemble des fragments solides est divisé de manière homogène en une première partie représentant 80 % en masse et une seconde partie représentant 20 % en masse. Le séchage est effectué sur la première partie, qui est ensuite enrobée par le liant introduit à 160 °C, après quoi on ajoute la seconde partie qui est à température ambiante et qui présente un taux d'humidité de 4 %.

Exemple 5

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**[0020]** Dans cet exemple, les fragments solides sont composés en masse de 2 % de fines calcaires, comme dans les exemples précédents, et de 98 % de granulats silico-calcaires de granulométrie 0/10 mm, la répartition granulométrique de ces derniers étant la suivante :

50	6/10	29
	4/6	28
	0/4	41

55 **[0021]** On procède comme dans l'exemple 3, à ceci près que les deux parties des fragments solides n'ont pas la même composition granulométrique, la première partie comprenant la totalité des fractions 6/10 et 4/6 et des fines et une quote-part de la fraction 0/4 correspondant à 16 % du total des fragments solides, et la seconde partie étant constituée par les 25 % restant de la fraction 0/4, présentant un taux d'humidité de 4 %. Par ailleurs, la quantité de liant est portée à 5,9 % de la masse des fragments solides.

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[0022] Le tableau ci-après donne pour chacun des exemples la température finale de l'enrobé et diverses caractéristiques de celui-ci. À titre de comparaison, les données correspondantes sont fournies pour des enrobés de référence obtenus à partir des mêmes composants par le procédé classique consistant à sécher la totalité des fragments solides à 160 °C avant l'enrobage.

5

Exemple	1	2	3	4	Référence
T finale (°C)	≈ 85	≈ 90	≈ 85	≈ 90	160
Température PCG	95	95	95	95	160
PCG %	7,1	6,8	7,1	8,8	9,3
% d'eau	0,3	0,3	0,4	0,2	-
% de vides	5,7	7,4	9,2	7,4	6,2
Orniérage	6,4	9,6	6,2	5,2	6,4
r/R	0,84	0,81	0,80	0,82	0,90

Exemple	5	Référence
T finale (°C)	≈ 90	160
Température PCG	95	160
PCG %	9,9	10,8
% d'eau	0,1	-
% de vides	8,0	5,8
Orniérage	5,7	6,4
r/R	0,70	0,88

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[0023] Dans le tableau :

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- "Température PCG" représente la température en °C à laquelle l'enrobé est porté pour réaliser l'essai PCG selon la norme française P 98-252,
- "PCG %" représente le pourcentage de vides obtenu pour 60 girations lors de l'essai à la presse à cisaillement giratoire (PCG) selon la norme française P 98-252,
- "% d'eau" représente la teneur en eau en masse de l'enrobé à l'issue de l'enrobage,
- "% de vides" représente le pourcentage de vides initial de l'éprouvette soumise à l'essai d'orniérage ,
- "Orniérage" représente le pourcentage de vides après 30 000 cycles lors de l'essai d'orniérage selon la norme française P 98-253-1,
- "r/R" représente le rapport r/R obtenu lors de l'essai Duriez selon la norme française P 98-251-1.

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#### Revendications

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1. Procédé pour fabriquer un enrobé comprenant des fragments solides enrobés d'un liant, dans lequel on sèche et on enrobe une première partie des fragments solides après quoi la partie restante des fragments solides, ayant conservé son humidité initiale, est ajoutée au mélange obtenu, **caractérisé en ce qu'**au moins ladite première partie des fragments solides contient des fines.
2. Procédé selon la revendication 1, dans lequel l'ensemble des fragments solides est divisé de manière homogène en ladite première partie et ladite seconde partie.
3. Procédé selon l'une des revendications 1 et 2, dans lequel lesdites première et seconde parties représentent respectivement 80 % et 20 % en masse des fragments solides.

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4. Procédé selon l'une des revendications précédentes, dans lequel l'étape de séchage est réalisée à une température supérieure à 100 °C, de préférence à une température d'environ 130 °C.
5. Procédé selon l'une des revendications précédentes, dans lequel une quantité contrôlée d'eau est ajoutée aux fragments solides entre les étapes de séchage et d'enrobage et/ou après l'étape d'enrobage.
6. Procédé selon l'une des revendications précédentes, dans lequel le liant est mis en oeuvre à une température comprise entre 100 et 200 °C, de préférence d'environ 160 °C, ou entre 30 et 90 °C dans le cas d'un liant sous forme d'émulsion.

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